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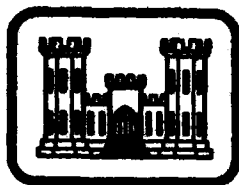UNCLASSIFIED   WES/TR/SL-81-1   NL

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DESIGN OF DOWELS FOR ANCHORING REPLACEMENT CONCRETE TO VERTICAL LOCK WALLS

by

Tony C. Liu, Terence C. Holland

Structures Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report presents the results of laboratory and field dowel pullout tests and shear transfer tests. These tests were conducted to evaluate the pullout performance of dowels and the influence of dowel spacing on the load-carrying capacity of the replacement concrete cast during rehabilitation of vertical walls in navigation locks.

The results of these tests and a review of the existing literature on the

(Continued)

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20. ABSTRACT (Continued).

Interface shear transfer and the dowel action mechanisms were the bases for the development of rational design criteria for dowels for anchoring replacement concrete to vertical lock walls. The design criteria include surface preparation, minimum dowel size, dowel spacing, and anchorage requirements. An example for designing dowels for a typical lock wall rehabilitation project is included.

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PREFACE

The study covered by this report was conducted in the Structures
Laboratory (SL), U. S. Army Engineer Waterways Experiment Station (WES),
under the sponsorship of the Office, Chief of Engineers, U. S. Army (OCE),
as a part of Civil Works Investigation Work Unit No. 31553, "Maintenance
and Preservation of Civil Works Structures." Messrs. James A. Rhodes
and Fred Anderson of the Structures Branch, Engineering Division, OCE,
served as Technical Monitors.

The study was conducted under the general supervision of
Mr. Bryant Mather, Chief, SL, and Mr. John Scanlon, Chief, Concrete Tech-
nology Division, and under the direct supervision of Mr. James E.
McDonald, Chief, Evaluation and Monitoring Group, SL. The tests were
conducted by Dr. Tony C. Liu, MAJ Terence C. Holland, and Messrs. J. T.
Peatross and F. W. Dorsey. Mr. W. B. Lee proportioned the concrete mix-
tures and fabricated all concrete test specimens. This report was pre-
pared by Dr. Tony C. Liu and MAJ Terence C. Holland.

The Commanders and Directors of the WES during this study and the
preparation and publication of this report were COL John L. Cannon, CE,
and COL Nelson P. Conover, CE. Technical Director was Mr. Fred R. Brown.

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CONTENTS

CONVERSION FACTORS, INCH-POUND TO METRIC (SI)
UNITS OF MEASUREMENT

Inch-pound units of measurement in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
inches	25.4	millimetres
kips (force)	4.448222	kilonewtons
kips (force) per square inch	6.894757	megapascals
pounds (force)	4.448222	newtons
pounds (force) per minute	4.448222	newtons per minute
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	0.006895	megapascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic yard	0.5933	kilograms per cubic metre
square feet	0.09290304	square metres
square inches	0.00064516	square metres
tons (2000 pounds, mass)	907.1847	kilograms

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DESIGN OF DOWELS FOR ANCHORING REPLACEMENT
CONCRETE TO VERTICAL LOCK WALLS

PART I: INTRODUCTION

Background

1. Seventy percent of the navigation locks operated by the Corps of Engineers are more than 20 years old; more than half (55 percent) of the locks are over 40 years old. Practically all of these structures will be in service well beyond their original design service life because of limited new construction starts. Consequently, a number of these structures are scheduled for rehabilitation within the next few years.

2. A significant portion of most navigation lock rehabilitation projects is work on the lock walls. Typical damage found on lock walls is shown in Figure 1.* The deteriorated concrete seen in the figures will be removed, and the lock walls will be restored to original dimensions by the placement of new concrete. A typical construction drawing showing resurfacing details is given in Figure 2.

3. The first step in the repair technique is to remove the surface concrete to a depth of 12-24 in.** by means such as drilling and blasting. Figure 3 shows a section of lock wall after concrete has been removed by the blasting technique.

4. The next step in the rehabilitation process is to install dowels in the walls. These dowels serve to position vertical and horizontal reinforcing steel in the replacement concrete and to anchor the replacement concrete to the existing wall elements. These dowels are usually reinforcing bars with a 90-degree bend (Figure 4). They may be

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 * The figures in this part of the report are from renovation work
    accomplished at Locks and Dam No. 3, Monongahela River, Elizabeth,
    Penn.
 ** A table of factors for converting inch-pound units of measurement
    to metric (SI) units is found on page 3.

a. Lock walls at monolith joint
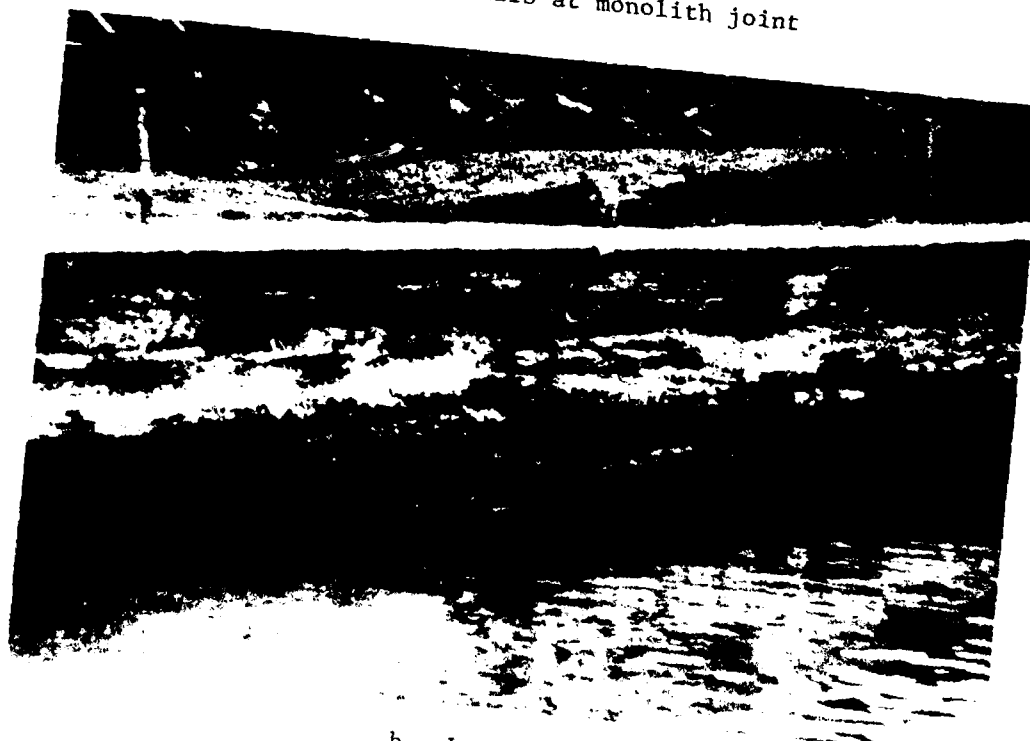
b. Lock walls

Figure 1. Typical deteriorated concrete in a navigation lock
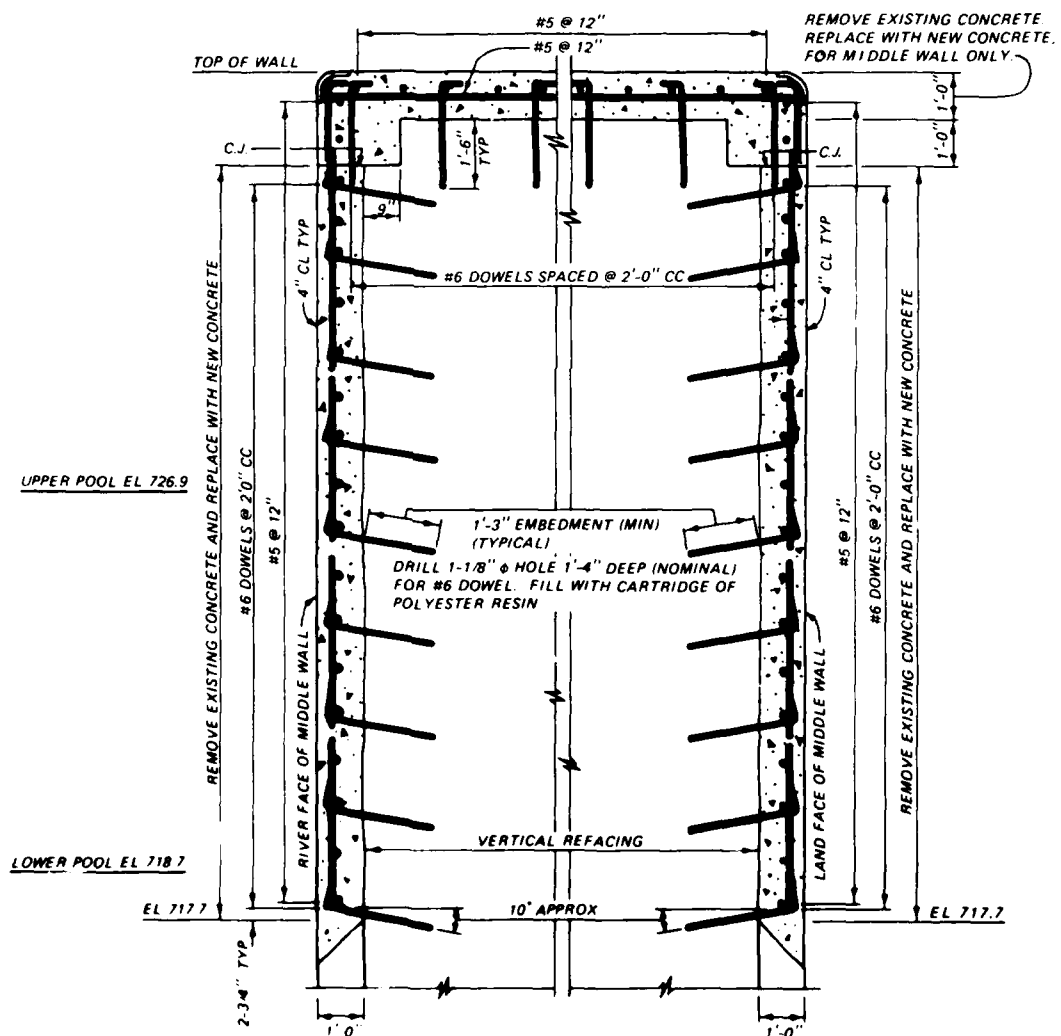
Figure 2. Renovation details, typical project

6
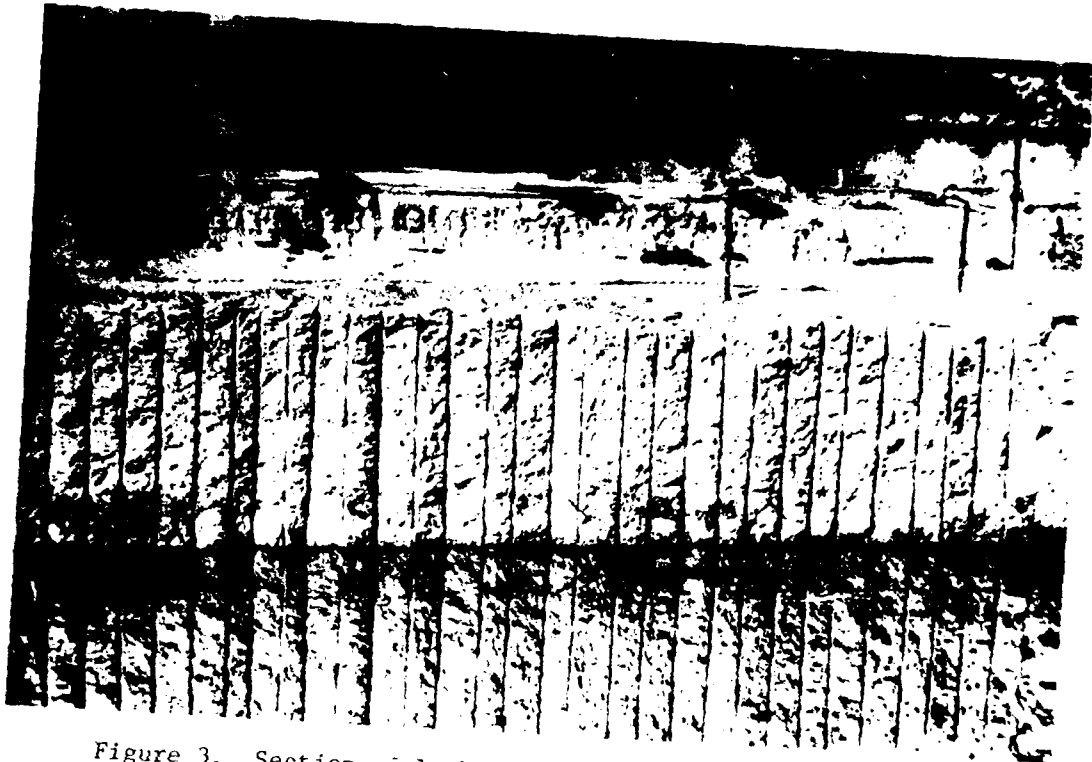
Figure 3. Section of lock wall after removal of deteriorated concrete by line drilling and blasting

Figure 4. Typical dowel, No. 6 reinforcing bar (ASTM 1978)

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anchored into the old concrete by the use of hydraulic-cement grout, epoxy resin, or polyester resin with a 15- to 3-in. embedment being typical. Figure 5 shows a polyester-resin cartridge system being used

Figure 5. Installation of dowels, polyester-resin cartridge system (Note: air wrench spins dowel, mixing components of resin cartridge; dowel held at proper position until resin sets)

to install dowels. Figure 6 shows a section of wall in which the installation of dowels has been completed.

5. Once the dowels are in place, reinforcing steel is placed, exterior forms are positioned, and concrete is placed to restore the walls to original dimensions.

6. The steps most open to question in the rehabilitation process have been the design and installation of the dowels. No engineering data were found upon which to base dowel size and spacing. As a result, a large number of dowels on close centers has usually been specified. Since installation of the dowels is very labor intensive, it turns out to be a very costly segment of the project.
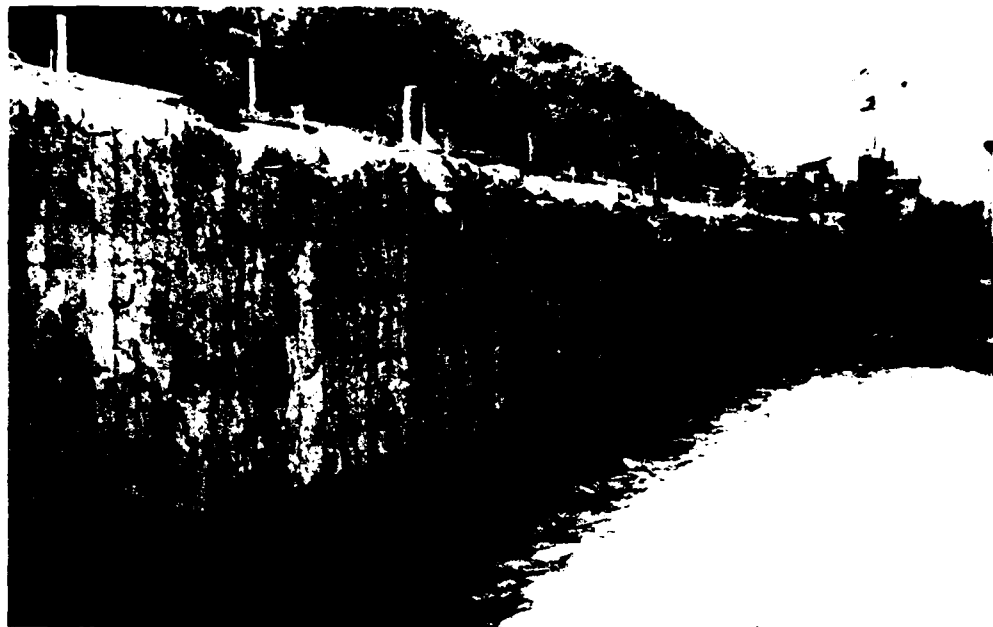
Figure 6. Section of lock wall with dowels installed, spaced 2 ft on
center in both directions (straight bars are form ties)

7. The question has been raised whether the typical size and
spacing of dowels currently specified for Corps projects are too con-
servative. A reduction in the number of dowels required for a project
could lead to significant cost savings. Conversely, if no reduction was
deemed feasible, engineering data would support that decision. The
present study was undertaken by the U. S. Army Engineer Waterways Experi-
ment Station (WES) to resolve these questions.

Objective and Scope

8. The objective of this investigation was to develop realistic
design criteria and engineering guidance for design and use of dowels
for anchoring new concrete to lock walls during rehabilitation. This
project is limited in scope to relatively thin sections of replacement
concrete that are cast in place and that can be expected to face normal
service conditions.

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Description of Study

9. This study consisted of five parts:

   a. Laboratory pullout tests of dowels anchored using polyester-resin cartridges.

   b. Field pullout tests of contractor-installed dowels (the same type of polyester-resin cartridges was used).

   c. Literature survey of load-transfer mechanism across concrete interface.

   d. Laboratory tests of load-carrying capacity across vertical joints that contain dowels representing various percentages of steel.

   e. Development of a design method and design guidance for dowels anchoring replacement concrete to vertical lock walls.

PART II: DOWEL PULLOUT TESTING

Laboratory Pullout Tests

Objective and scope

10. The purpose of the laboratory pullout tests was to develop
test apparatus and procedures for conducting in situ pullout tests on
vertical lock walls, and to evaluate the effects of embedment lengths on
the pullout resistance of No. 6 reinforcing bars.

11. A large concrete block was used as a test bed. Holes for the
embedment of reinforcing bars were drilled into the test block and the
bonding agent was commercially available polyester-resin cartridges. A
total of eight pullout tests were conducted in the laboratory.

Test block

12. The test block in which the reinforcing bars were embedded
was cast in 1972 as part of a mass concrete slipform construction pro-
gram conducted at the WES (Saucier 1974). The block measured 3 by 6 by
10 ft and contained 6-in. maximum size limestone aggregate. The physi-
cal properties of aggregates are shown in Table 1. The concrete mixture
was proportioned to have a cementitious medium content of approximately
260 lb/cu yd, an air content of 4.7 percent, a slump of 1-3/4 in., and
a fly ash content of 35 percent by volume of the cementitious medium.
Detailed mixture proportions are given in Table 2.

13. Five 6-in.-diameter cores (Figure 7) were drilled from the
test block using a diamond bit. These cores were taken from the center
and the four corners of the test block to obtain representative samples.
Exact locations of the cores are shown in Figure 8.

14. Five 6- by 12-in. specimens were prepared from the cores and
were tested for compressive strength and tensile splitting strength in
accordance with CRD-C 14-73* and CRD-C 77-72, respectively. The results
of these tests are presented in Table 3. The average compressive

---

* All CRD-C test methods are published in Handbook for Concrete and
  Cement (WES 1949).

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Table 1

Physical Properties of Fine and Coarse Aggregates

and Gradings of Coarse Aggregates

Parameter		Size Range				
	Fine	No. 4 to 3/4 in.	3/4 to 1-1/2 in.	1-1/2 to 3 in.	3 to 6 in.	Combined Coarse Aggregate*
Physical Properties						
Bulk specific gravity, saturated surface dry	2.66	2.72	2.71	2.70	2.69	
Absorption, %	1.6	0.5	0.5	0.3	0.3	
Cumulative Percent Passing						
Sieve						
6 in.					100	100
5 in.					87	95
4 in.				100	65	86
3 in.				93	35	73
2 in.				47	8	53
1-1/2 in.			100	14	2	46
1 in.		100	98	3	0	29
3/4 in.		97	30	2	0	21
1/2 in.		67	1	0	0	15
3/8 in.		35	0	0	0	8
No. 4		5	0	0	0	0

* Proportions of the four sizes of coarse aggregate were as follows: 3- to 6-in., 40%; 1-1/2- to 3-in., 18%; 3/4- to 1-1/2-in., 20%; No. 4 to 3/4-in., 22%.

Table 2

Mixture Proportions

Material	Solid Volume, cu ft	Mass, Saturated Surface Dry lb/cu yd
Portland cement	0.933	183
Fly ash	0.502	78
Fine aggregate	4.904	814
Coarse aggregate	17.389	2932
Water	2.435	162
Air	0.837	--
	27.000	4169
Slump, in.*	1-3/4	
Air content, %*	4.7	

* Portion passing 37.5-mm (1-1/2-in.) sieve.

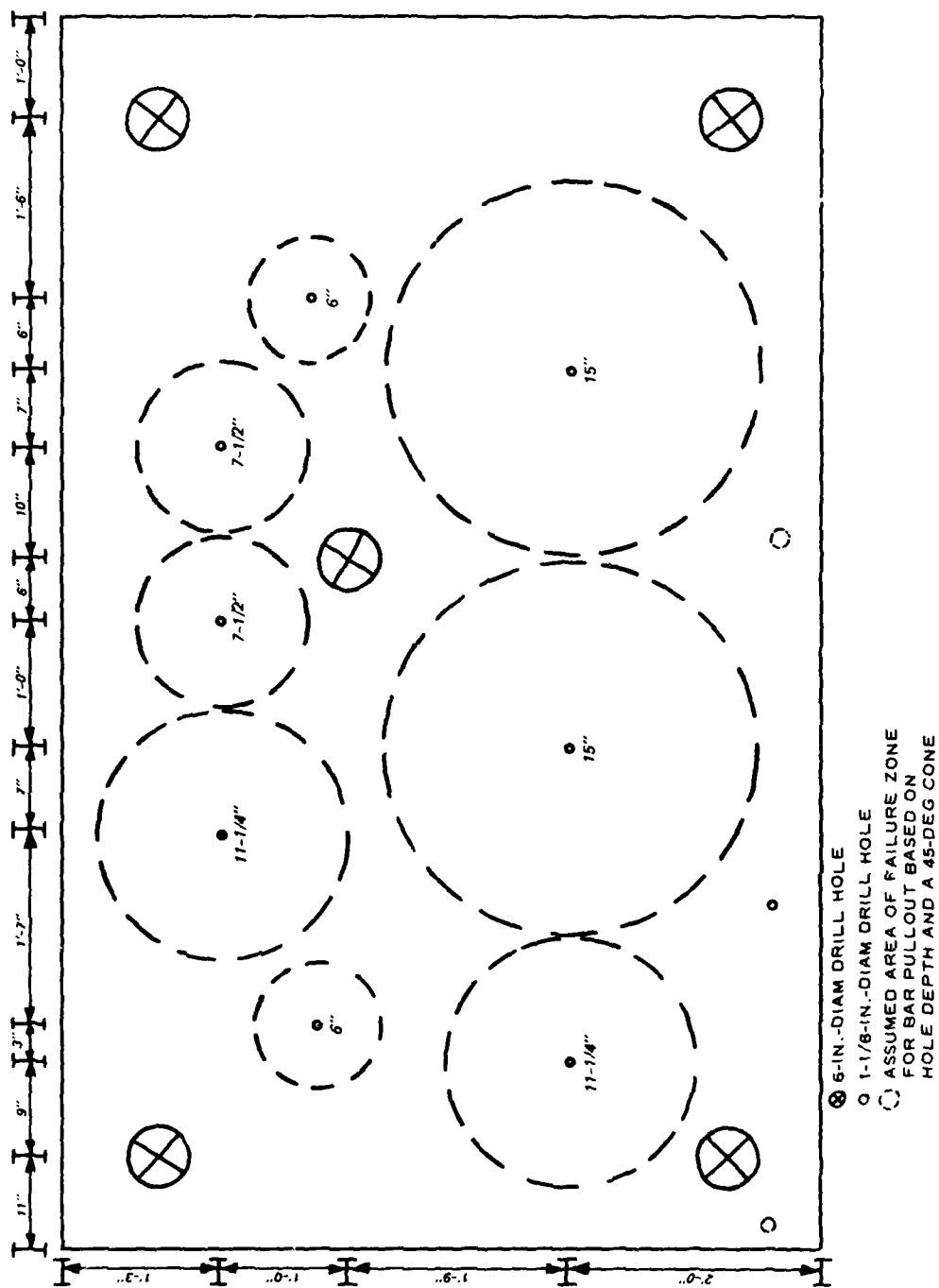Figure 7. Test block samples, 6-in.-diameter cores

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Figure 8. Drill hole layout

Table 3

Results of Strength Tests

Core	Dimensions in.	Failure Load lb	Compressive Strength psi	Tensile Splitting Strength psi
1	5.8 x 12.3	60,000	2270	--
2	5.8 x 12.4	71,000	2690	--
3	5.8 x 12.3	68,000	2580	--
4	5.8 x 12.0	49,600	--	455
5	5.8 x 12.1	47,600	--	430

strength and the tensile splitting strength were 2510 psi and 440 psi, respectively.

Drill hole layout

15. The layout for the 1-1/8- and 6-in.-diameter drilled holes is shown in Figure 8. The 1-1/8-in.-diameter holes were drilled using a pneumatic rotary-percussive drill (Figure 9). These holes were drilled deep enough to embed No. 6 reinforcing bars at embedment lengths equal to 6.0, 7.5, 11.25, and 15.0 in. (embedment length to nominal bar diameter ratios, L/D = 8, 10, 15, and 20, respectively). To simulate the field conditions, all 1-1/8-in.-diameter holes had a 10-degree inclination from the horizontal.* These drilled holes were spaced to allow for a possible 45-degree conical failure of the concrete.

Reinforcing bars

16. Standard No. 6 deformed reinforcing bars were used. Two bars were tested in uniaxial tension in accordance with the applicable portions of CRD-C 501-76. The average yield strength was 47,270 psi. A typical stress-strain curve of the reinforcing bars is shown in Figure 10.

Resin cartridges

17. Commercially available polyester-resin cartridges (1 in.

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* The slight inclination was used to prevent loss of the bonding agent.

Figure 9. Pneumatic rotary-percussive drill

* FAILURE STRENGTH - 72,720 PSI,
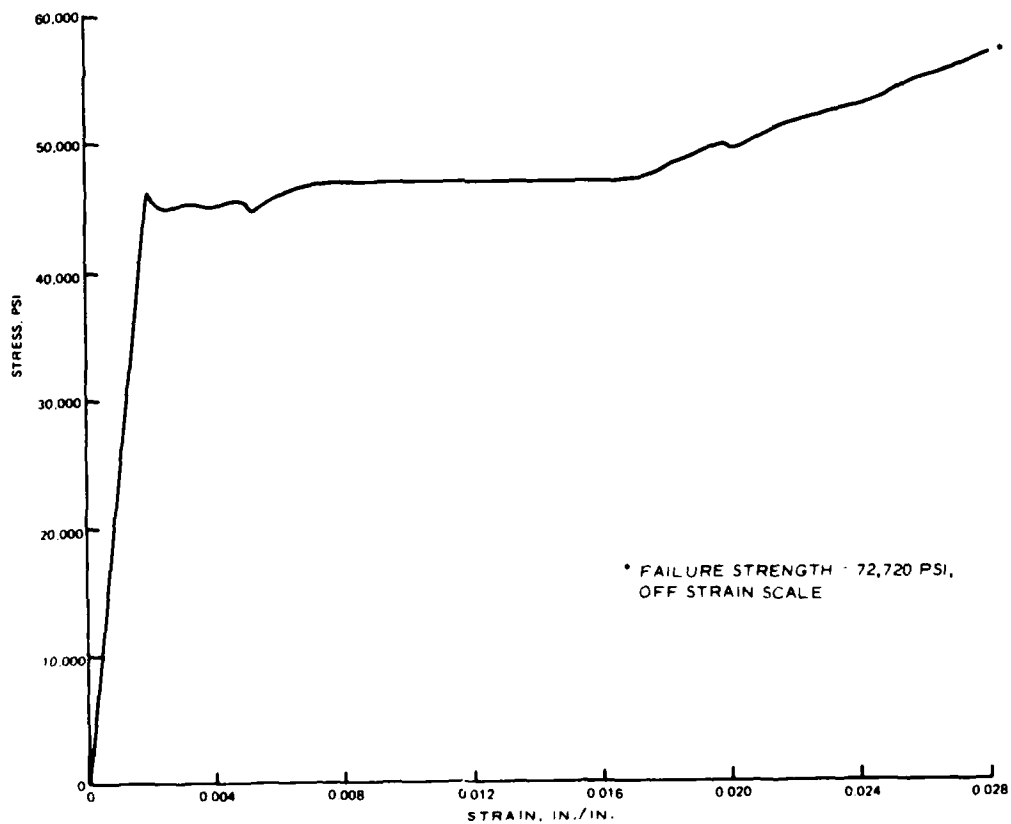 OFF STRAIN SCALE

STRAIN, IN./IN.

Figure 10. Typical stress-strain curve of reinforcing bars

diameter by 12 in. long) were used (Figure 11). The cartridges consisted
of a pack containing a polyester-resin component with a catalyst. The
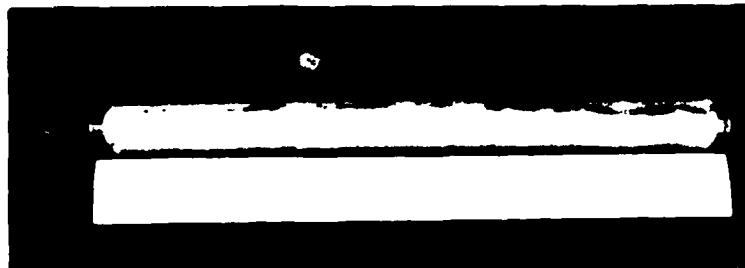
Figure 11. Typical polyester-resin cartridge

components are isolated from each other by a physical-chemical barrier
that prevents reaction between the components until required. No reac-
tion takes place until the reinforcing bar is rotated through the
cartridge which mixes the components and initiates the curing action.
The mixed resin fills the volume (annulus) around the reinforcing bar
and bonds firmly to the concrete within minutes.

Reinforcing bar embedment

18. The procedures for embedding the reinforcing bars in the
drilled holes were as follows:

a. Air flush the drilled holes to remove all debris and dust.

b. Insert a polyester-resin cartridge.

c. Force the reinforcing bar into the hole breaking the
cartridge.

d. Couple a pneumatic drill (Figure 12) to the reinforcing
bar* and rotate the bar into the hole at 200 to 450 rpm.

e. Stop inward movement when the reinforcing bar reaches the
desired embedment length, and continue rotating the bar
for 15 to 20 sec to thoroughly mix the resin system.

f. Stop rotation and uncouple the drill from the reinforcing
bar. (The bar was firmly bonded when the resin set in few
minutes.)

----

* The exposed end of the bar was threaded and was fitted with a
hexagonal nut.

Figure 12. Pneumatic drill used to insert and
spin the reinforcing bar

Test apparatus and procedures

19. The test apparatus used to conduct the pullout tests is shown
in Figure 13. The reaction frame was constructed with 9-in. channel sec-
tions. The clear span of the reaction frame is 2 ft 2 in. A 60-ton,
hollow plunger jack in combination with an electric pump (Figure 14) was
used to apply the axial load to the reinforcing bar. To account for the

Figure 13. Test apparatus, laboratory pullout tests
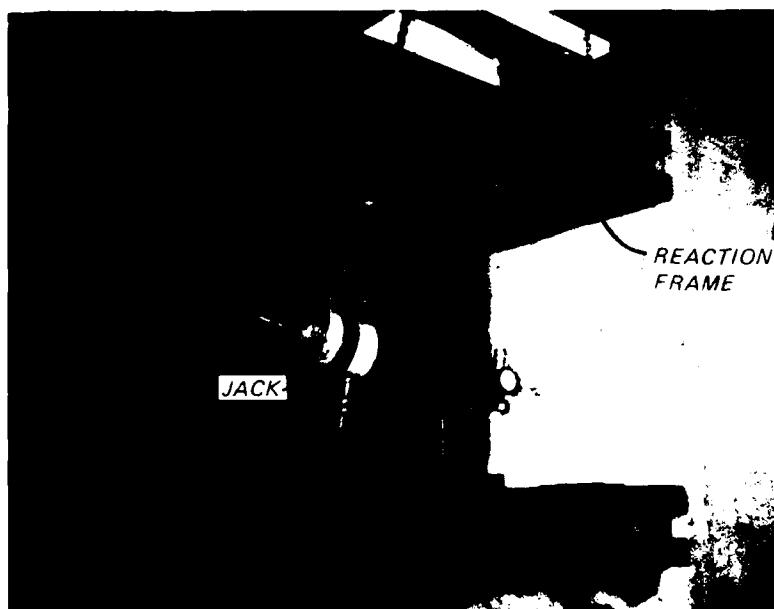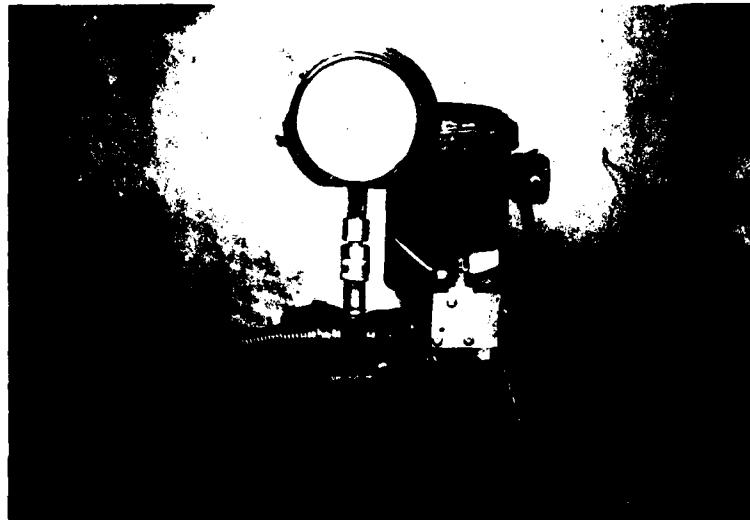
18

Figure 14. Electric pump, laboratory pullout tests

10-degree inclination of the reinforcing bar, a special steel shim plate (Figure 15) was inserted between the jack and reaction frame. The jacking system was calibrated, and as the load was applied to the bar, a 10,000-psi pressure gage was monitored. From the pressure gage readings,
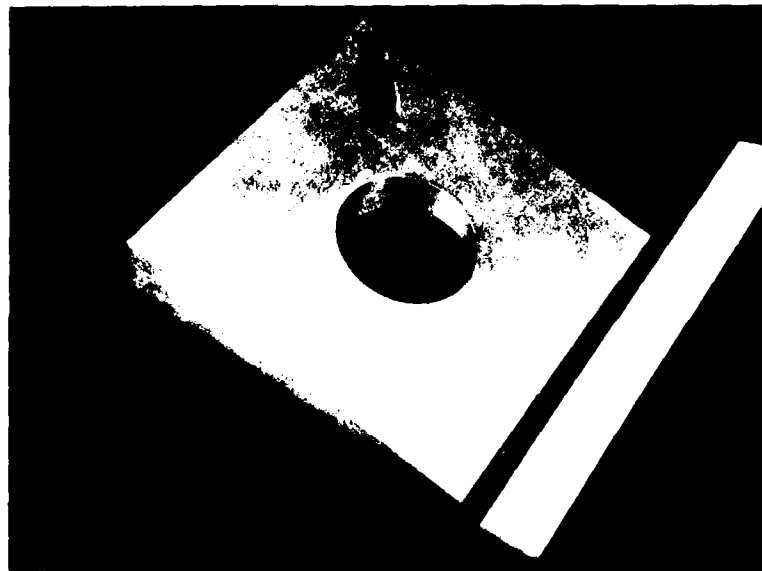
Figure 15. Steel shim plate

19

the jack load was determined for any increment of pressure.

20. A 0.7-in. prestressing chuck (Figure 16) held the exposed end
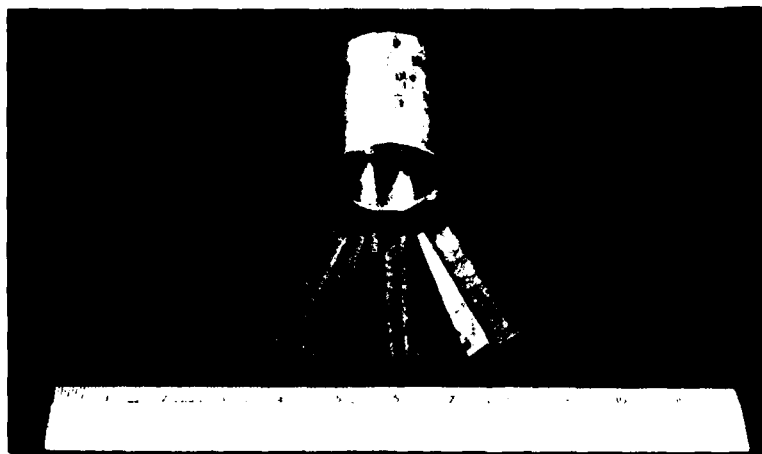
Figure 16. Prestressing chuck

of the reinforcing bars. The chuck performed adequately, evident by the
fact that no yielding was observed in this region of the bar.

21. The relative displacement of a bar to the concrete block was
measured by the use of two dial gages. The gages were mounted on a
cross arm that was attached to the bar adjacent to the concrete block
surface. Displacement readings were taken at regular increments of
jacking pressure.

22. After the polyester resin had cured for approximately 30 min-
utes, the bar was tested for pullout resistance. A particular test was
terminated when no additional load could be applied to the reinforcing
bar because of either slippage or excessive elongation.

Test results and discussions

23. The results of the laboratory pullout tests are presented in
Appendix A and are summarized in Table 4. The average pullout loads for
embedment lengths of 6.0, 7.5, 11.25, and 15.0 in. were 15.3, 18.1,
20.8, and 22.2 kips, respectively. The relation between embedment
length and average pullout load is shown in Figure 17.

24. All reinforcing bars yielded when the embedment length was
greater than 11.25 in. (L/D = 15). The yielding was evident when the

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Table 4

Laboratory Pullout Test Results

Specimen No.	Embedment Length in.	L/D	Pullout Force kips	Type of Failure
1	6.0	8	16,680	Slippage
2	6.0	8	13,900	Slippage
3	7.5	10	16,680	Slippage
4	7.5	10	19,460	Slippage
5	11.25	15	19,460	Slippage
6	11.25	15	22,240	Yielding
7	15.0	20	22,240	Yielding
8	15.0	20	22,240	Yielding

rust on the bar began flaking off and when the bar would not support additional load. The yielding was also evident when the axial load reached 20.8 kips, which is the yield strength of the bar.

25. Slippage along the concrete-resin interface generally caused failure when the embedment length was less than 11.25 in. A typical bar after pullout is shown in Figure 18.

26. The load-displacement curves for the laboratory pullout tests are shown in Figures 19-22. The initial slopes of these curves were approximately the same. However, the total displacements of the bars with a shorter embedment length were generally larger than those of the bars with a longer embedment length. The average total displacements were 0.14 in. and 0.07 in. for bars with embedment lengths of 6 in. and 15 in., respectively.

27. No concrete failure was observed for the embedment lengths investigated.

Figure 17.   Relation between embedment length and
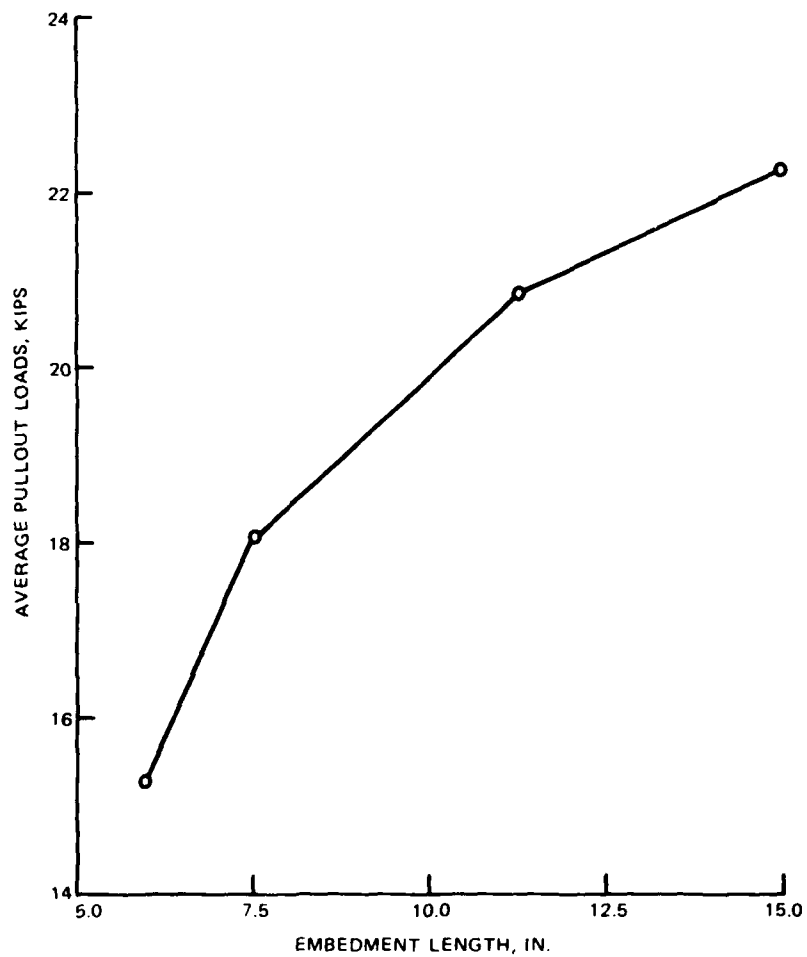average pullout load

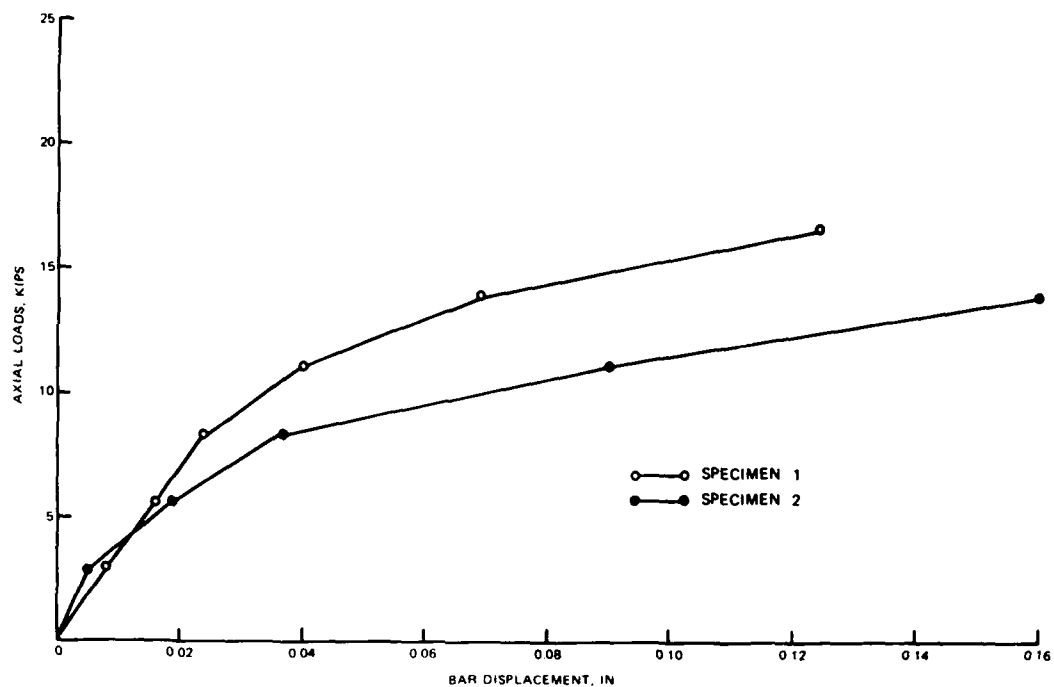Figure 18.   Typical bar after pullout

22

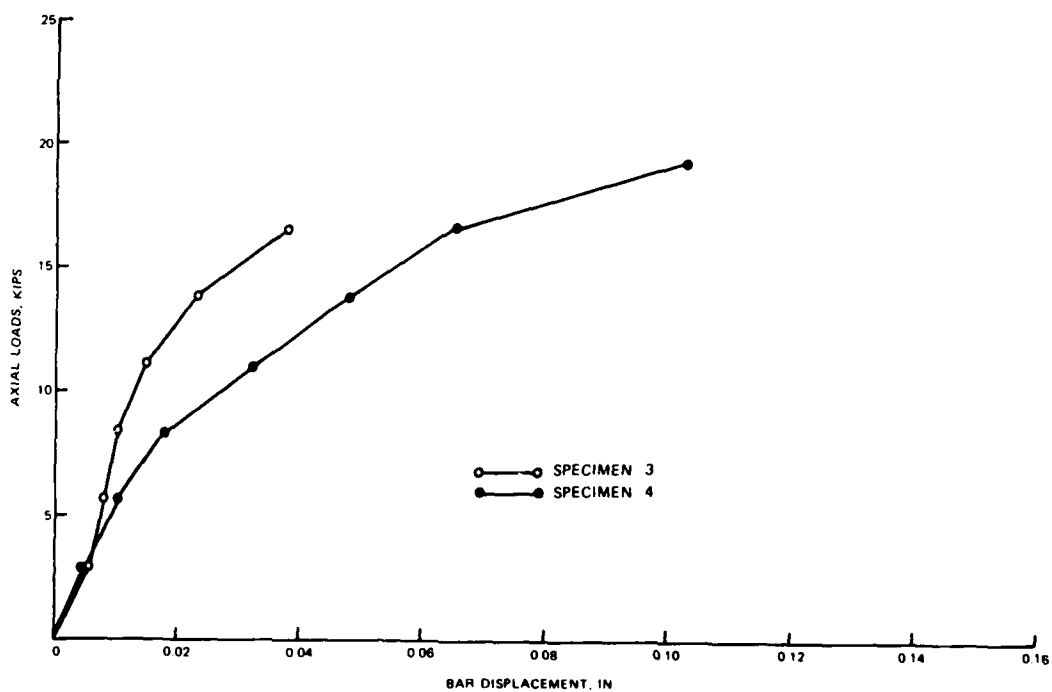Figure 19. Axial load-bar displacement curves, 6-in. embedment length

Figure 20. Axial load-bar displacement curves, 7.5-in. embedment length

23
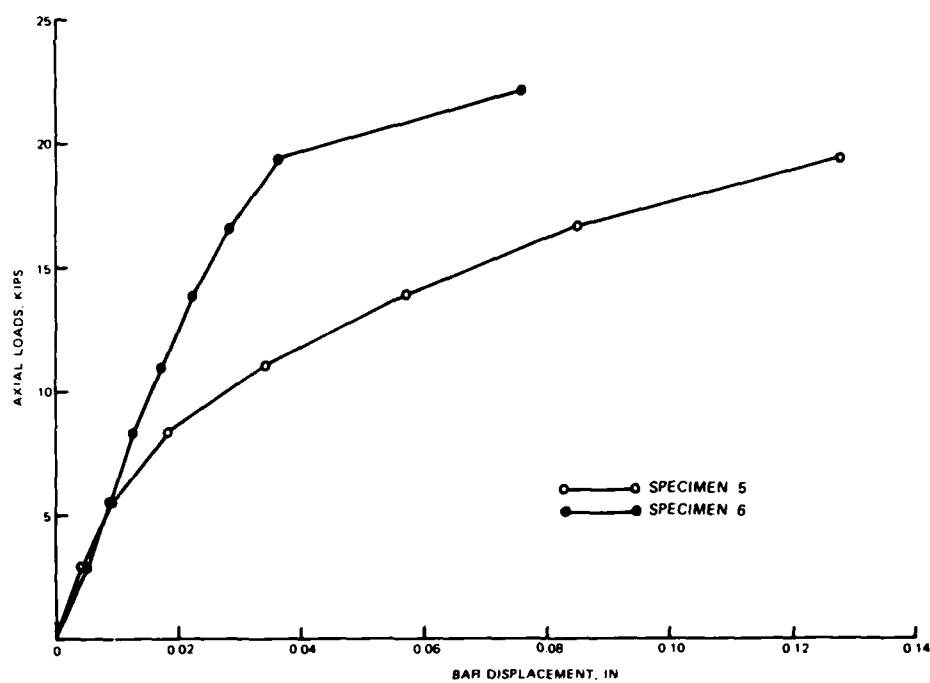
Figure 21.   Axial load-bar displacement curves, 11.25-in.
embedment length
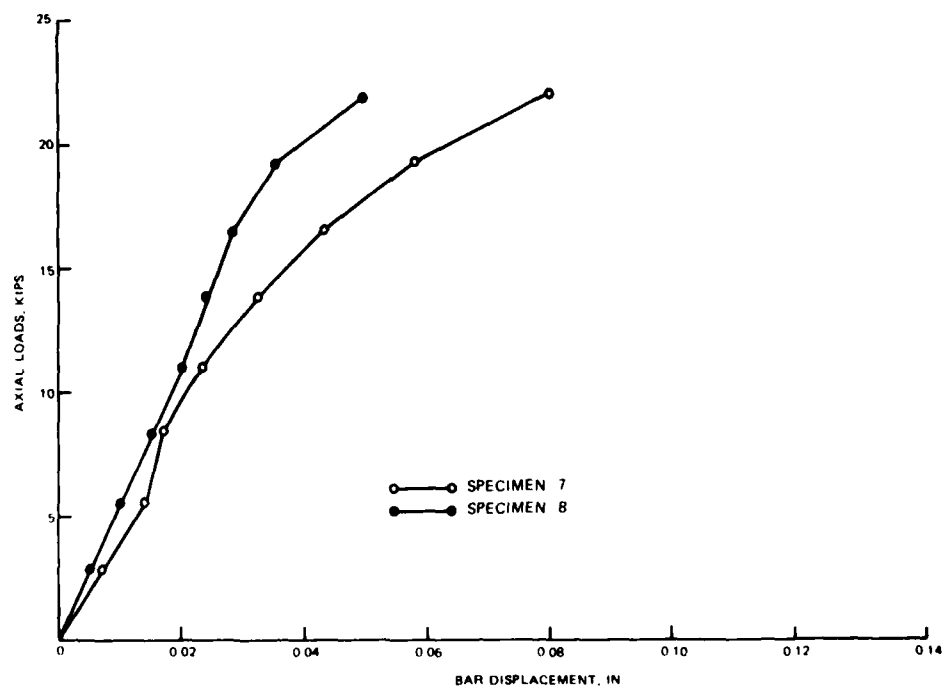
Figure 22.   Axial load-bar displacement curves, 15-in.
embedment length

24

Field Pullout Tests

Objective

28. The objective of the field pullout tests was to determine the pullout performance of dowels installed under field conditions by a contractor working on a major rehabilitation project. At the site selected,* the dowels were being installed by the use of the polyester-resin cartridge system and the basic technique that had been used for the laboratory pullout tests.

Test apparatus

29. The original test plan envisioned the use of the laboratory pullout test apparatus. This approach was determined to have two problems:

 a. Use of the laboratory pullout test apparatus would require installation of special dowels long enough to be gripped by the jack; day-to-day installations could not be tested. This caused concern that the bars to be tested might receive special attention during placement and would not therefore be representative.

 b. Site inspection indicated that the test apparatus probably could not be supported in the proper position for testing. The top of the lock walls did not have enough room for a crane to operate. All of the test equipment would therefore have to have the capability of being placed in and operated from a small boat.

30. The test plan was revised to allow a random selection of actual contractor-placed dowels for testing. This modification was achieved by revisions to the test equipment that are described below.

31. Several modifications were made in the test apparatus (the revised apparatus is shown in Figure 23) for the site pullout tests.

 a. The reaction frame was eliminated, which allowed the reaction force to be transmitted directly to the wall surrounding the dowel. This was determined to be a reasonable approach since none of the bars tested in the laboratory had shown a concrete cone mode of failure.

 b. The hollow core jack was replaced by two flat jacks mounted on a steel plate. This significantly reduced the

---

* Locks and Dam No. 3, Monongahela River.

25

② LOCK WALL

① ③

DOWEL

10 DEG

TO
HYDRAULIC
PUMP

LEGEND

① PRESTRESSING CHUCK

② STEEL PLATE WITH
TWO FLAT JACKS

③ INCLINED STEEL PLATE
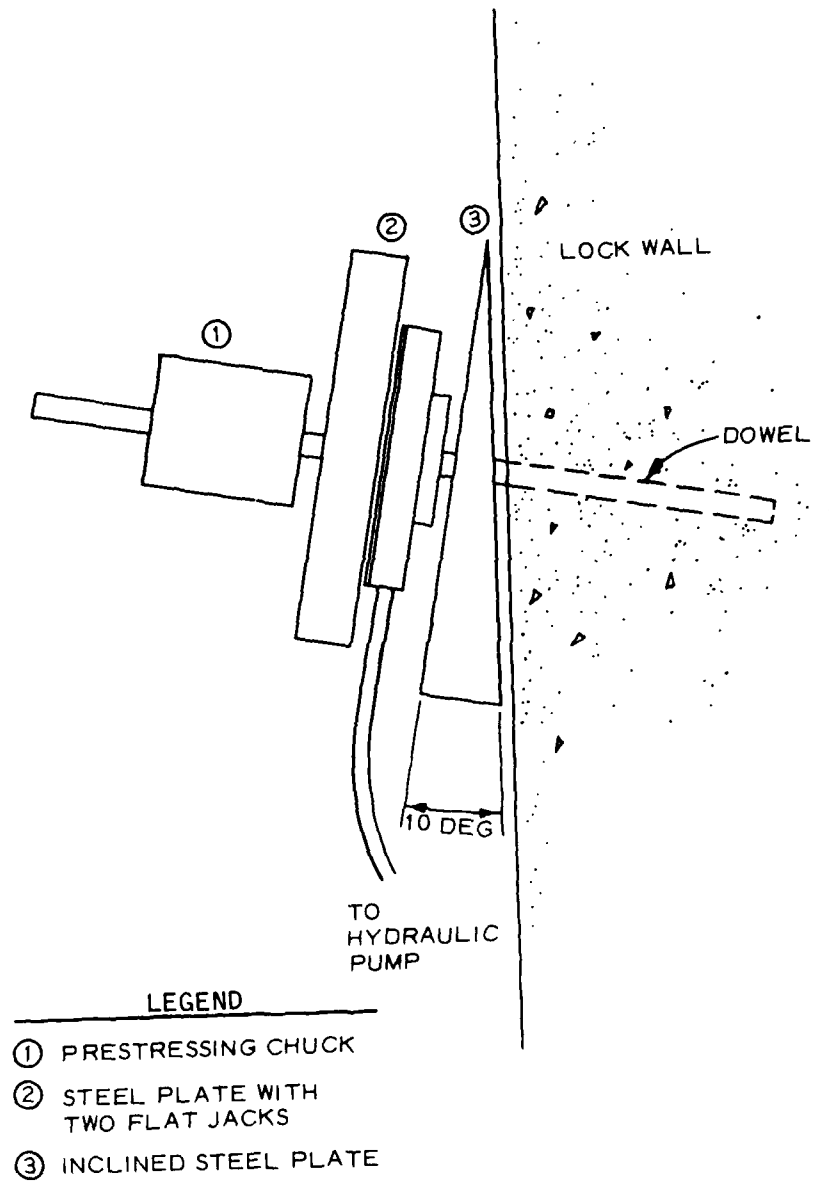
Figure 23. Modified testing apparatus, field
pullout tests (schematic)

26

length of bar required for testing. Figures 24 and 25 show the steel plate with mounted flat jacks.

c. The inclined plate and the prestressing chuck used in the laboratory tests were retained for the field tests (see Figure 23).

d. A manually operated hydraulic pump was used to power the two jacks (Figure 26). The jacks, pump, and gage were calibrated using a laboratory test machine. Table 5 shows the gage pressure to bar stress calibration chart developed and used during the testing.

Figure 24. Modified testing apparatus, lock wall pullout tests (Note: plate reversed for photo)

Figure 25. Assembled testing apparatus with pre-stressing chuck gripping dowel (Note: jacks are in operating position on back side of plate)
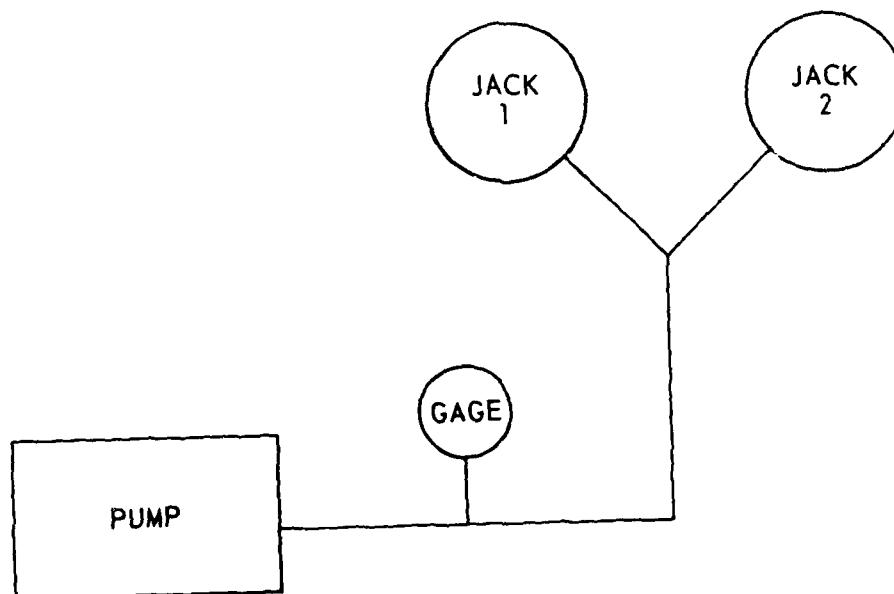
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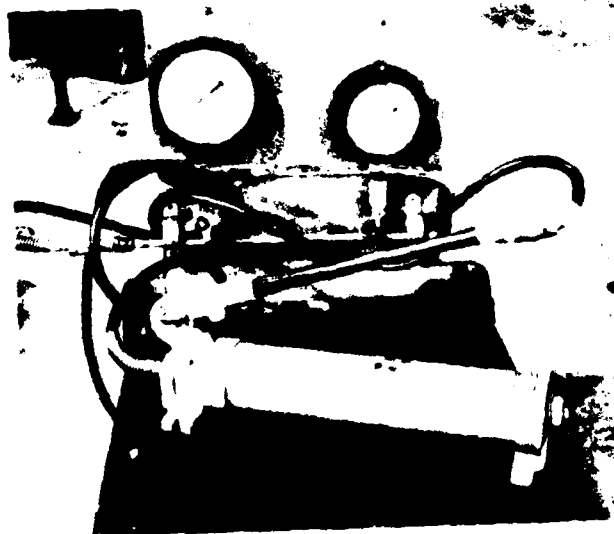
b. Photo

Figure 26. Hydraulic pump and pressure gage

Table 5

Calibration of Two Flat Jacks

Gage Pressure psi	Load, lb	Stress in No. 6 Bar* psi
500	2,333	5,302
1000	4,633	10,529
1500	6,767	15,379
2000	9,100	20,681
2500	11,183	25,416
3000	13,367	30,380
3500	15,450	35,114
4000	17,600	40,000
4500	19,617	44,584
5000	21,700	49,318
5500	23,633	53,711
6000	25,650	58,295
6500	27,633	62,802

* Area of No. 6 bars = 0.44 sq in.

Test procedure

32. The procedure used to perform the pullout tests at the site was follows:

a. Random dowels were selected from those inside the river chamber and those on the riverside of the middle wall downstream of the river chamber. Figure 27 shows test locations and monolith numbers.

b. The bends in the ends of the dowels were straightened using a come-along and a cheater pipe technique. This was necessary to provide enough dowel to grip even with the flat jack apparatus.

c. Surface irregularities in the vicinity of the dowel to be tested were reduced by hand to provide a bearing surface as flat as possible.

d. The pullout apparatus was mounted on the bar. Loading was done in gage pressure increments of 500 psi. Each
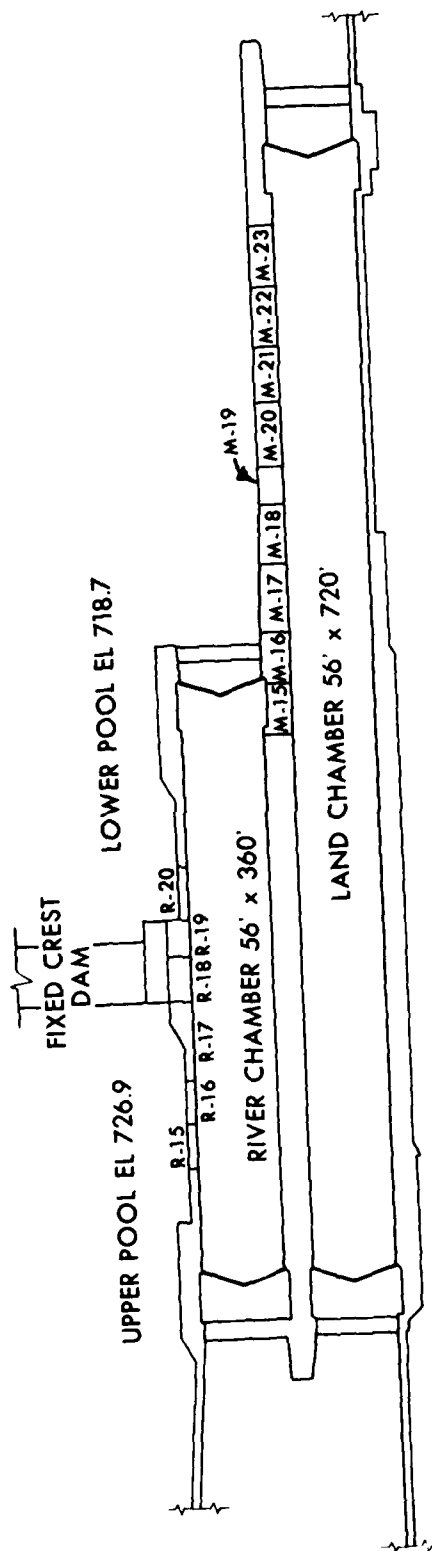
Figure 27. Site plan and test locations, Locks and Dam No. 3, Monongahela River

increment was held for approximately 30 sec before the
load level was increased.

e. Bars were normally loaded twice in each test. The first
loading would exhaust jack travel at a relatively low
load because of localized crushing of protruding concrete
beneath the inclined wedge. The initial load was then
released, and a new grip was obtained on the bar. Load-
ing was then again accomplished in increments as described
above.

f. Measurements of actual dowel pullout were not made. The
dowels were loaded until a given load could no longer be
held, indicated by loss of pressure on the gage, or until
the approximate yield range of the steel was reached.

Results and discussion

33. Data from the pullout tests are presented in Table 6. None
of the bars failed to hold at least the design stress of 40,000 psi.

34. Two dowels were returned to the WES for uniaxial tensile
strength determination in accordance with CRD-C 501-76. Data from these
tests are in Table 7.

35. Several of the dowels at the site held stresses much higher
than that of the average yield stress of the two samples returned to the
WES. This is attributed to possible use of higher than Grade 40 steel
for a portion of the dowels.

36. The exact cause of failure could not be determined for those
dowels unable to hold a load (Table 6). The yield of the steel had
probably been reached, since no evidence of pullout of the entire dowel
stub-resin assembly was noted.

37. A significant number of dowels on both the outside of the
center wall and the inside of the river chamber had been damaged (bent)
by the impact of work barges since the time of installation. Test
specimens 3 and 4 were in this condition. Although pullout performance
was apparently not affected, significant extra effort would be required
if in fact all such dowels were straightened before the remainder of
the reinforcing steel was placed.

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Table 6

Results of Dowel Pullout Tests, Locks and Dam No. 3, Monongahela River

Bar No.	Location*	Description	Test Loading	Remarks
1	Monolith M20, riverside of center wall below river chamber. Elevation 729 ft	Normal embedment length.**	Held 5,000-psi gage pressure = 49,300 psi stress in steel	
2	Monolith M20, riverside of center wall below river chamber. Elevation 729 ft	Normal embedment length.	Held 4,500-psi gage pressure = 44,500 psi stress in steel	Would not hold 5,000-psi gage pressure
3	Monolith M21, riverside of center wall below river chamber. Elevation 727 ft	Normal embedment length. Bar badly bent due to impact loading; straightened for test.	Held 4,500-psi gage pressure = 44,500 psi stress in steel	Slow drop-off at 5,000-psi gage pressure
4	Monolith M20, riverside of center wall below river chamber. Elevation 723 ft	Normal embedment length. Bar badly bent due to impact loading; straightened for test.	Held 4,500-psi gage pressure = 44,500 psi stress in steel	
5	Monolith M18, riverside of center wall below river chamber. Elevation 725 ft	Normal embedment length.	Held 5,000-psi gage pressure = 49,300 psi stress in steel	Slow drop-off at 5,500-psi gage pressure
6	Monolith M18, riverside of center wall below river chamber. Elevation 725 ft	Normal embedment length. No epoxy visible at front of hole. Bar felt loose when shaken by hand.	Held 6,000-psi gage pressure = 58,300 psi stress in steel	No sign of any movement in bar

(Continued)

* Lower pool elevation, 721 ft; inside river chamber, 718 ft.
** Normal embedment length was 15-18 in.

32

Table 6 (Concluded)

Bar No.	Location	Description	Test Loading	Remarks
7	Monolith M18, riverside of center wall below river chamber. Elevation 729 ft	Normal embedment length.	Held 4,500-psi gage pressure = 44,500 psi stress in steel	Would not hold 5,000-psi gage pressures; could not determine cause of failure
8	Monolith R15, river wall inside river chamber. Elevation 722 ft	Normal embedment length.	Held 6,500-psi gage pressure = 62,800 psi stress in steel	
9	Monolith R15, river wall inside river chamber. Elevation 726 ft	Normal embedment length. Located in apparent horizontal construction joint which was damp.	Held 6,500-psi gage pressure = 62,800 psi stress in steel	
10	Monolith R16, river wall inside river chamber. Elevation 723 ft	Less than normal embedment due to indentation in wall at bar location.	Held 6,000-psi gage pressure = 58,300 psi stress in steel	
11	Monolith R17, river wall inside river chamber. Elevation 722 ft	Normal embedment length.	Held 5,500-psi gage pressure = 53,700 psi stress in steel	
12	Monolith R18, river wall inside river chamber. Elevation 721 ft	Normal embedment length. Possibility that epoxy cartridges used in this area were outdated; used with manufacturer's approval.	Held +6,500-psi gage pressure = 62,800 psi stress in steel	

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Table 7

Dowel Tensile Test* Data, Locks and

Dam No. 3, Monongahela River

Avg Diameter in.	Area sq in.	Yield		Failure	
		Load lb	Stress psi	Load lb	Stress psi
		Bar 1			
0.627	0.309	15,800	51,130	25,800	83,500
		Bar 2			
0.647	0.329	15,200	46,200	26,600	80,850

* Dowels tested 28 September 1979.

PART III: SHEAR TRANSFER

Literature Survey

38. An understanding of the shear-transfer mechanism across an
interface between new and old concrete is of critical importance in
dowel design. The shear-transfer mechanism, which acts by combined ac-
tion of aggregate interlock and friction and dowel action, has been
studied by many investigators, mainly through tests simulating connec-
tion and construction joints in precast and cast-in-place concrete. A
summary of the existing literature will be reviewed in this section to
evaluate the performance of the combined action of the interface shear-
transfer and the dowel-action mechanisms.

39. Birkeland and Birkeland (1966) present a shear-friction hy-
pothesis to describe the maximum shear force that can be transferred
across a precast concrete connection. The model employed is shown in
Figure 28. The shear load applied to the specimen produces tangential
and normal displacements at the shear plane. The normal displacements
will develop axial tensile stresses in the reinforcement crossing the
crack, which will induce vertical compressive stresses on the concrete.
The resistance to sliding will then be provided by the frictional force
generated by the vertical compressive stresses in the concrete. From
horizontal equilibrium (Figure 28)

$$V = T_n \tan \alpha \tag{1}$$

where

V = shear force

T_n = induced vertical compressive force

$\tan \alpha$ = coefficient of friction between the two concrete surfaces

40. The ultimate shear capacity will be developed when yield
strength is reached in the reinforcement crossing the shear plane.
Hence, the ultimate shear force can be expressed by

35

$$V' = A_s f_y \tan \alpha \tag{2}$$

where

    V' = total ultimate shear force

    A_s = total cross sectional area of reinforcement across interface

    f_y = yielding strength of reinforcing steel
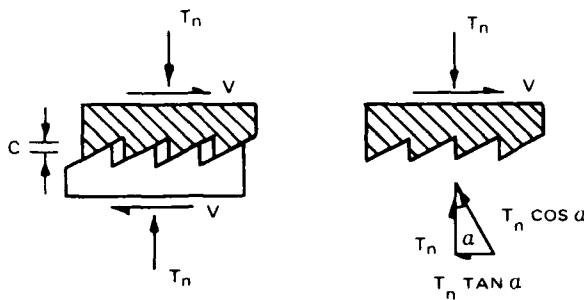
Figure 28. Shear-friction model (after Birkeland and Birkeland 1966)

41. The following values for the coefficient of friction were suggested by Birkeland and Birkeland (1966):

Monolithic concrete	1.7
Artificially roughened joints	1.4
Ordinary construction joints	0.8-1.0

42. It must be noted that Equation 2 is valid for conditions in which failure is attained by yielding of the reinforcement crossing the crack. Thus, proper development lengths should be provided to the reinforcement, and the concrete must be well confined.

43. Mast (1968) also compared the shear friction theory to experimental results of composite beams and corbel tests. Similar to the results presented by Birkeland and Birkeland (1966), the shear friction equation for corbels with horizontal tension can be expressed as

$$V' = (A_s f_y - T_h) \tan \alpha \tag{3}$$

where

T_h = horizontal tensile force at ultimate

44. Mattock and his co-workers (Mattock 1974a; Mattock 1974b; Mattock 1974c; Mattock 1974d; Mattock 1974e; Mattock 1976; Mattock and Hawkins 1972) have conducted several investigations into the ultimate shear strength of initially cracked and uncracked concrete. The setups employed for different specimens and loadings are shown in Figure 29. All the specimens were loaded by pure shear on the shear plane until failure occurred by yielding of the reinforcement. These investigations

Figure 29. Different test specimens used by Mattock (after Mattock 1974d and Mattock and Hawkins 1972)

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studied the effect on the ultimate shear strength of different percentages and arrangements of reinforcement, the concrete and reinforcement strengths, the presence of direct stresses acting parallel and tra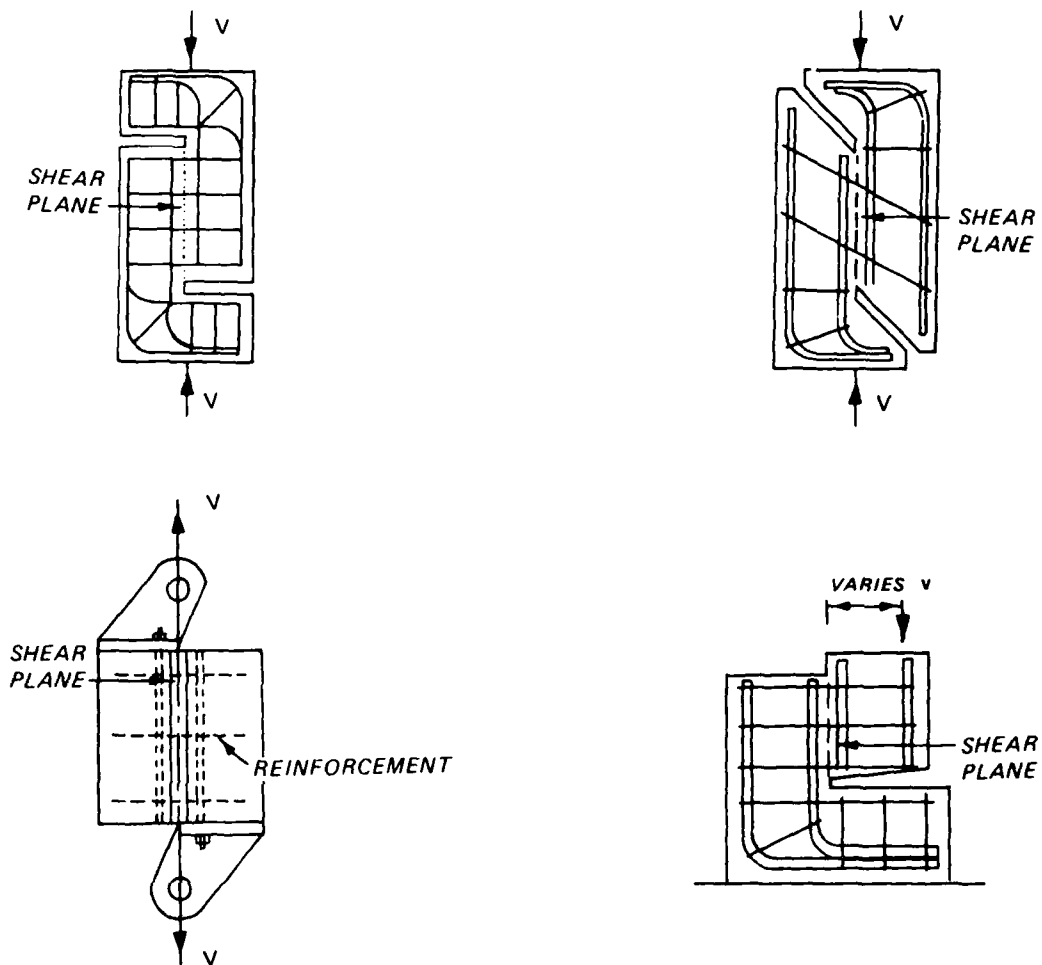nsverse to the shear plane, the presence of moments and tensile forces normal to the shear plane, the aggregate type, presence of construction joints on the shear plane, and the effect of cyclic shear stresses.

45. These tests indicated a basic difference in behavior between initially cracked and uncracked concrete specimens. The uncracked specimens indicated that a series of small diagonal cracks formed near the shear plane at high shear stresses. A concrete strut between the small, inclined cracks transferred the shear stresses with little relative displacement occurring on either side of the crack. The cracked specimens, however, indicated that large relative displacements on the shear plane were required to resist the applied shear loads.

46. As shown in Figure 30, the ultimate shear stress increases almost linearly with the index, ρf_y,* from a finite value for ρf_y equal to zero to a limit dependent on the concrete strength for high ρf_y values. Strengths are consistently greater with a monolithic shear plane than with a precracked shear plane. The strength for a precracked shear plane decreases rapidly with decreasing ρf_y for ρf_y values less than 200 psi. For ρf_y lying between A and A' and B in Figure 30, failure is relatively gentle and is due to a breakdown of the concrete after yielding occurs in the reinforcement crossing the shear plane. For ρf_y values lying between B and C, failure occurs abruptly before the reinforcement yields. The failure loads in this region are similar to uncracked and precracked specimens.

47. The Mattock tests also indicated that the behavior of a construction joint with rough interfaces is similar to that of the initially cracked specimens. If the interface is smooth, the interface shear-transfer mechanism is eliminated and the shear stress is transferred

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* ρ = reinforcement ratio = $\dfrac{\text{area of dowel reinforcement}}{\text{area of the shear plane}}$
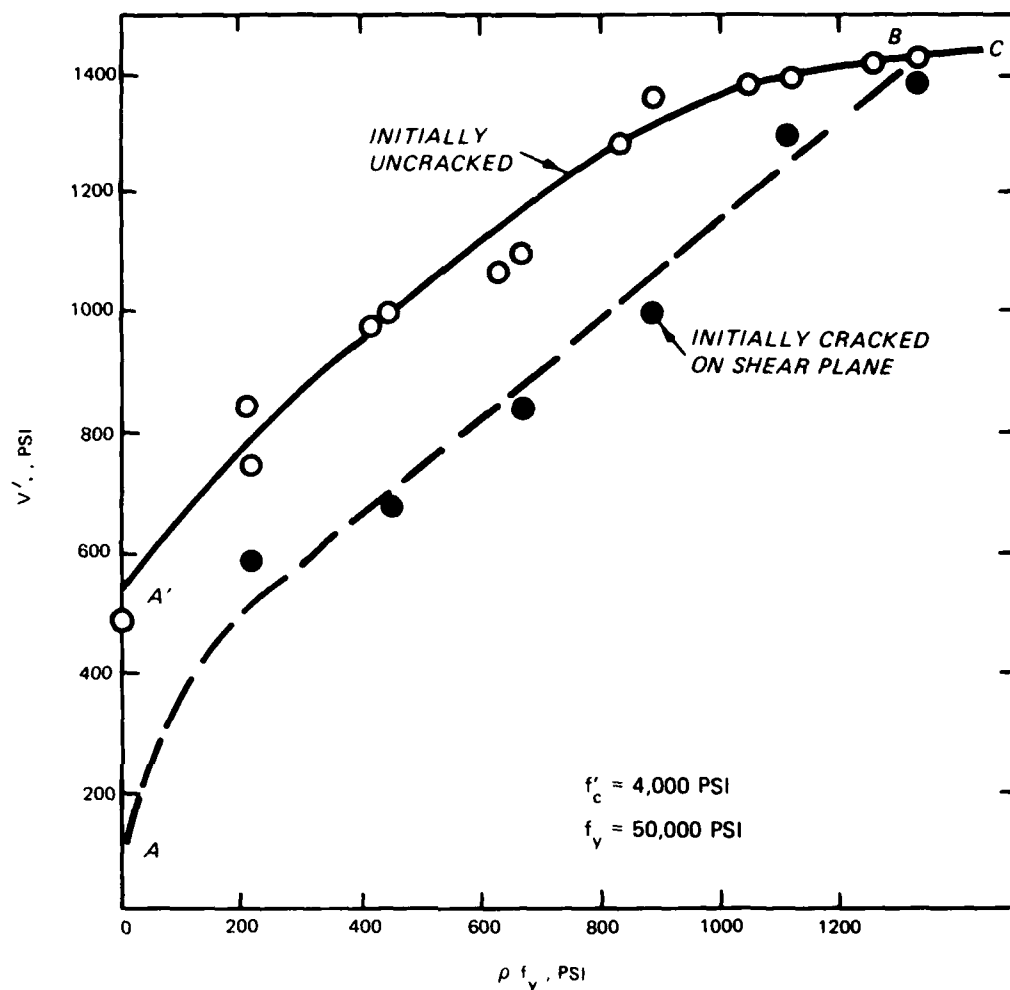
Figure 30. Variation of shear strength with reinforcement
parameter ρf_y (after Hofbeck, Ibrahim, and Mattock 1969)

mainly by the dowel-action mechanism. The ultimate shear strength for
rough surfaces can be predicted by the shear friction theory using a
friction coefficient of 1.4 in Equation 2. The ultimate shear strength
for smooth surfaces is given by the following equation:

$$V' = 0.58 \, A_s f_y \tag{4}$$

48. The Mattock tests further showed that the ultimate shear

strength of the specimens subjected to cyclic loading averaged 83 percent of the shear strength measured for the monotonically loaded specimens. The application of successive loading cycles increased the shear displacement at maximum load and reduced the shear stiffness of the specimen.

49. In tests in which the parameter ρ was varied by varying both the bar spacing and the bar size, Mattock and Hawkins (1972) and Hofbeck, Ibrahim, and Mattock (1969) found no marked variation in the effect of ρf_y. The effect of the yield strength of the dowels was similarly found to be properly reflected by the parameter ρf_y.

50. Paulay, Park, and Phillips (1974) conducted an investigation to identify the principal mechanisms of shear resistance across horizontal construction joints. The test specimen and the setup employed are shown in Figure 31. The construction joint, reinforced with steel
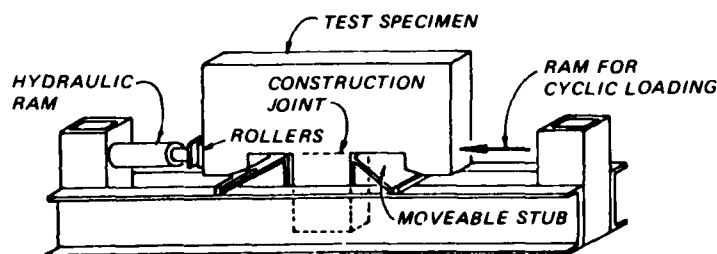
Figure 31. Test setup used by Paulay (after
Paulay, Park, and Phillips 1974)

percentages ranging between 0.31 and 1.23 percent, was tested to failure by the application of either monotonic or cyclic shear stresses. The other variable investigated was the surface preparation of the construction joint. Test results indicated that the initial slopes of the load-displacement curves were almost identical for the variables tested. However, the maximum shear stress increased with an increase in surface roughness and reinforcement percentage. Paulay also observed that for low steel percentages, failure was attained by yielding of the reinforcement crossing the construction joints. Failure for high steel percentages occurred by concrete being crushed at the shear plane. The shear friction theory with a coefficient of friction of 1.4 predicts conservatively the ultimate shear stresses measured experimentally.

Summary

51. Based on the survey of several experimental investigations presented above, the shear-transfer mechanism across an interface between new and old concretes may be hypothesized as follows.

a. The shear forces initially are transferred through the uncracked interface by bond. When a crack forms at the interface, the shear forces are transferred by the combined action of aggregate interlock and friction and by dowel action.

b. When shear acts along a crack, slip of one crack face occurs with respect to the other. If the crack faces are rough and irregular, this slip is accompanied by separation of the crack faces. This separation will stress the dowel crossing the crack, which in turn will provide a clamping force across the crack faces. The applied shear is then resisted by friction between the crack faces and by dowel action of the reinforcement crossing the crack.

52. The shear-friction theory proposed by Birkeland and Birkeland (1966) provides a lower bound to the experimental data available on shear test specimens. The shear-friction method of calculation assumes that all shear resistance is due to friction between the crack faces. Therefore, a reasonable value of the coefficient of friction for the shear-friction equation must be developed so that the calculated shear strength will be in reasonably close agreement with test results.

Laboratory Shear Transfer Tests

Objectives

53. The objectives of the laboratory shear transfer tests were to develop the value of the coefficient of friction to be used in the shear-friction equation and to examine the influence of dowel spacing on the load-carrying capacity of the replacement concrete in a lock wall renovation. These tests were accomplished using laboratory-cast blocks to represent the old and new concretes. Dowels of various diameters were used with the blocks to represent the spacing variations in prototype structures. Table 8 shows a comparison of laboratory dowel sizes and prototype dowel spacing.

Table 8

Conversion of Test Dowel Spacings

to Prototype Spacings

Test		Prototype
No. 3 bars at 24 in.	=	No. 6 bars at 48 in.
No. 4 bars at 24 in.	=	No. 6 bars at 36 in.
No. 5 bars at 24 in.	=	No. 6 bars at 29 in.
No. 6 bars at 24 in.	=	No. 6 bars at 24 in.

54. A slight modification in prototype dowel spacing can lead to significant reductions in the number of dowels required. Table 9 shows the number of dowels required for a hypothetical rehabilitation project using the spacings tested in the laboratory.

Table 9

Dowel Requirements for Various Spacings

in a Lock Wall Renovation*

Spacing, C-C, in.	Area per Dowel, sq ft	Dowels Required	Percent**
24	4	3000	100
29	5.84	2055	69
36	9	1333	44
48	16	750	25

* Hypothetical lock wall; assume lock wall is 600 ft long by 20 ft high (12,000 sq ft).
** Percent of number of dowels required for 24-in., c-c spacing.

Test materials

55. The same concrete mixture was used for both old and new portions of the test blocks. The nominal, 4500-psi mixture contained 3/4-in. maximum sized aggregate. Table 10 presents the details of the concrete mixture proportions.

56. Steel samples representing each of the four dowel sizes used

Table 10

Mixture Proportions

Material	Solid Volume cu ft	Mass Saturated Surface Dry lb/cu yd
Portland Cement, Type I	2.544	500.0
Fine Aggregate, Limestone*	9.264	1560.7
Coarse Aggregate, Limestone**	10.037	1703.6
Water	5.155	321.7
	27.000	4086.0

* Specific gravity, 2.70; percent absorption, 0.7.
** Maximum size, 3/4 in.; specific gravity, 2.72; percent absorption, 0.4.

in the tests were tested in uniaxial tension in accordance with CRD-C 501-76. The results of this testing are presented in Table 11.

Test blocks

57. The test blocks were designed to provide a contact area of 576 sq in. between the old and new concretes. To provide this area, the blocks were sized as shown in Figure 32. The dimensions of the old and new portions of each block were identical. Each portion (old or new) contained approximately 5.7 cu ft of concrete and weighed approximately 850 lb.

Table 11

Steel Test Data

Bar	Diameter in.	Area sq in.	Yield Load lb	Yield Stress psi	Failure Load lb	Failure Stress psi
No. 6	0.665	0.34	18,800	55,300	33,200	97,600
No. 5	0.580	0.26	15,900	61,200	24,600	94,600
No. 4	0.451	0.16	9,150	57,200	13,050	81,600
No. 3	0.295	0.07	4,000	57,100	6,150	87,900
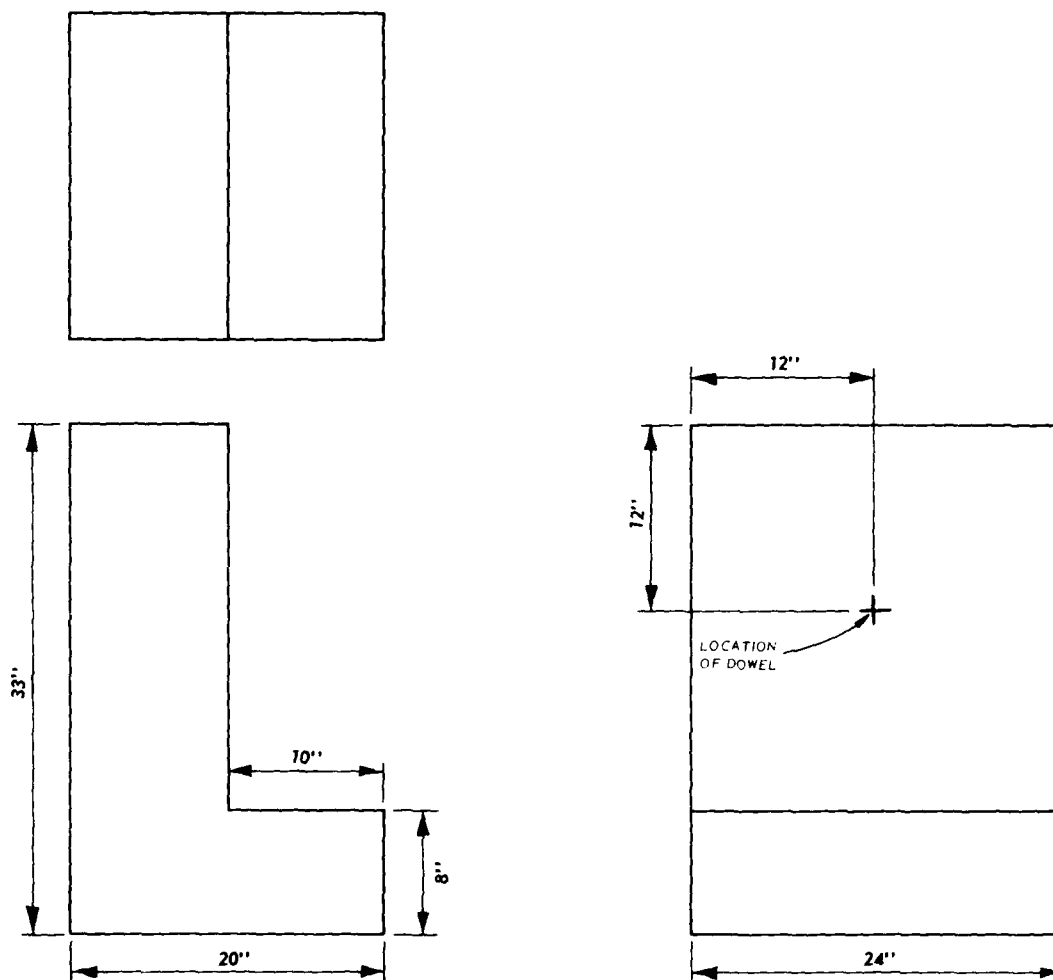
Figure 32.   Dimensions, old and new specimens

58.   Reinforcement was placed in the old and new blocks (Figure 33).
All reinforcement except the dowels linking the old and new portions of
the blocks was the same for the tests.

59.   The old portion of the blocks was cast in a position so that
the contact surface between the old and new concretes would be available
for finishing (Figure 34).   After the concrete had stopped bleeding, a
retarder was applied to the contact surface.   The forms were removed
approximately 24 hr after the blocks were cast, and the contact surface
was cut with a low-pressure water jet to remove paste and to provide a

44
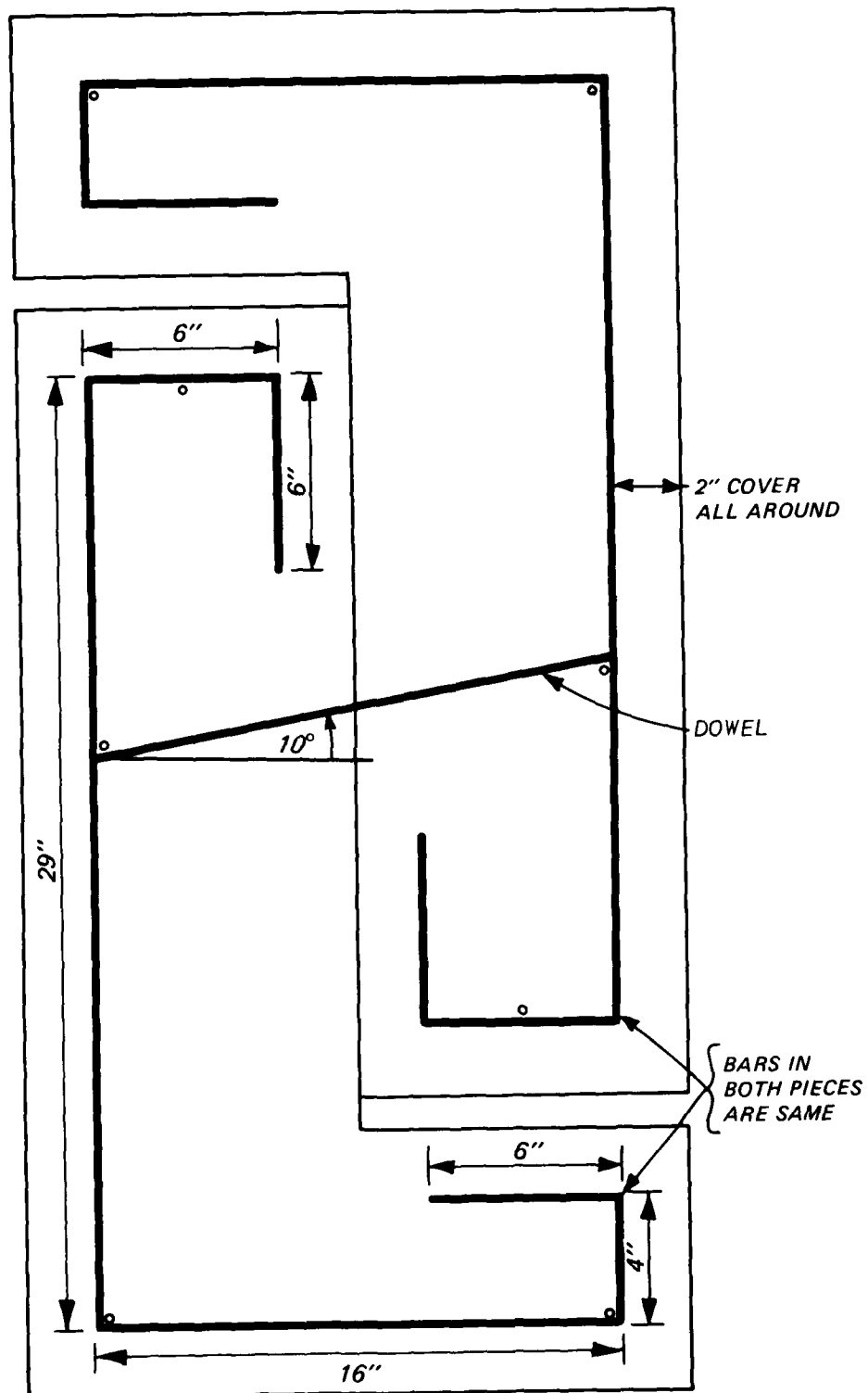
NOTE: DIMENSIONS ARE TO OUTSIDE OF REBAR.

Figure 33. Details, bent No. 4 bar
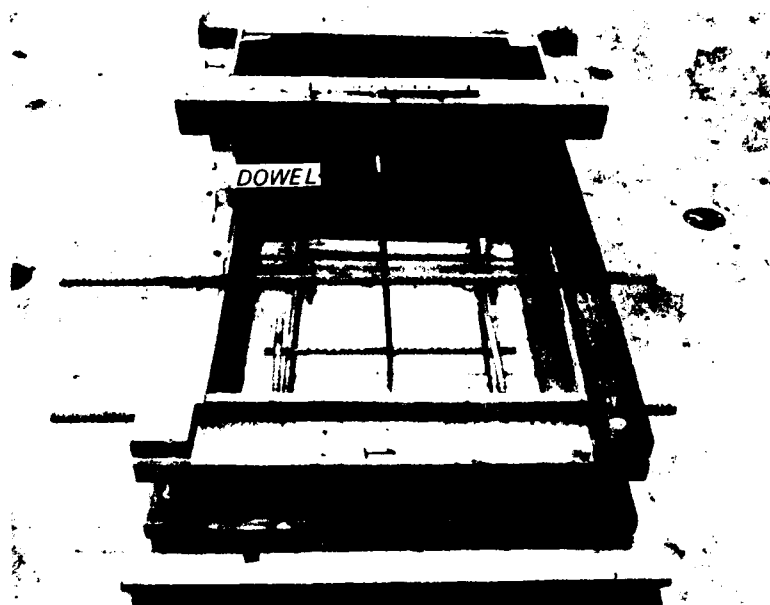
Figure 34. Form for casting old portion of shear block

roughened surface. The blocks were then stored at 100 percent relative humidity for a minimum of 28 days.

60. The old concrete blocks were removed from the fog room after at least 28 days, and were turned to the position shown in Figure 35. Forms and reinforcing steel were then placed for the casting of the new concrete. A compressible material was used to cast a void between the blocks to allow for movement during testing. After the forms were removed the block assemblies were stored for 28 days at 100 percent relative humidity. Figure 36 shows a completed block assembly.

Test procedure

61. The compressible material between the blocks, shown in Figure 36, was removed prior to the testing. Testing was accomplished using a 440,000-lb testing machine that applied loading at a rate of 25,000 lb/min. Loading was applied by the use of a plate and bar assembly (Figure 37) mounted on top of the blocks. Figures 38-40 show a block in place in the testing machine. The blocks were loaded to failure (i.e. separation of the new concrete from the old concrete).

NEW

1-INCH
COMPRESSIBLE
MATERIAL
(REMOVED FOR
TESTING)

1-INCH
COMPRESSIBLE
MATERIAL
(REMOVED FOR
TESTING)

OLD

Figure 36. Assembled old and new concrete
block ready to test

LOCATION
OF DOWEL

Figure 35. Old concrete block prior to
casting new concrete

Figure 37.   Close-up of load-transfer plate and
bar assembly
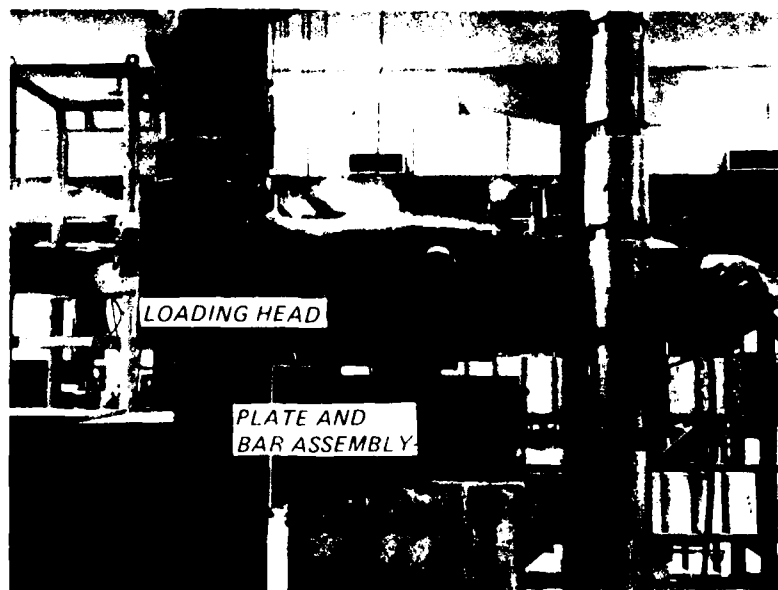
Figure 38.   Close-up of loading head nearing load-
distribution plate and bar on top of shear block

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Figure 39. Front view of shear block in testing machine prior to loading

Figure 40. Side view of shear block in testing machine during loading

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Results and discussion

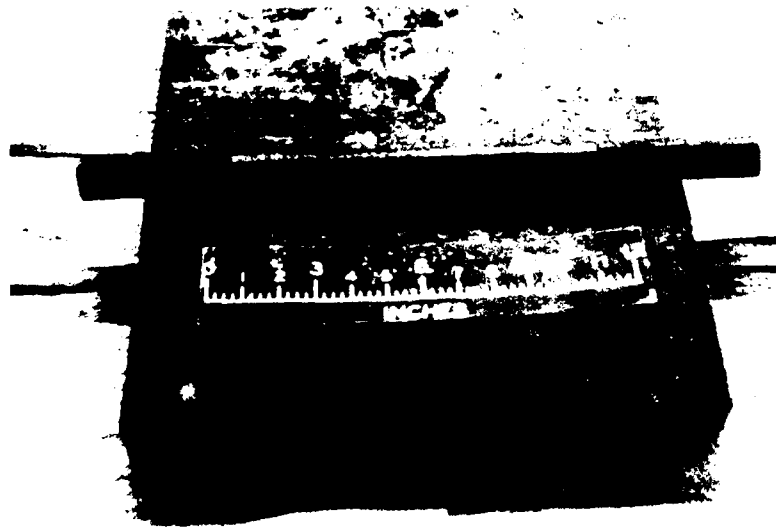62. Data from the shear-transfer tests are presented in Appendix B and summarized in Table 12. The specimens that did not contain dowels showed an average shear strength of 196 psi. The average ultimate shear strengths were 203, 234, 242, and 233 psi, respectively, for specimens containing No. 3, 4, 5, and 6 dowels.

Table 12

Summary of Shear Test Data

	Dowel Size				
	No. 0	No. 3	No. 4	No. 5	No. 6
Dowel area, sq in.	0.00	0.11	0.20	0.31	0.44
Percentage of steel	0.000	0.019	0.035	0.054	0.076
ρf_y (w/f_y = 40,000 psi)	0.00	7.64	13.89	21.53	30.56
Old concrete					
Compressive strength, psi*	5550	6140	6470	5080	5420
Flexural strength, psi*	840	825	925	805	830
New concrete					
Compressive strength, psi*	4560	5120	5480	5110	4850
Flexural strength, psi*	815	755	760	800	715
Average shear stress, psi	196	203	234	242	233
Number of specimens in averages	2	2	2	2	3

* Weighted averages based upon number of test specimens from each batch of concrete.

63. The effect of the dowels within the range of the percentage of steel (Table 12) tested did not appear to increase significantly the load-carrying capacity of the blocks. Instead, the bond of the new to the old concrete appeared to have been much more significant than the amount of steel present.

64. Figure 41 plots the average ultimate shear stresses against the values of ρf_y. There appears to be a slight upward trend in the data that shows a small but increasing contribution from the dowels as dowel size increases.
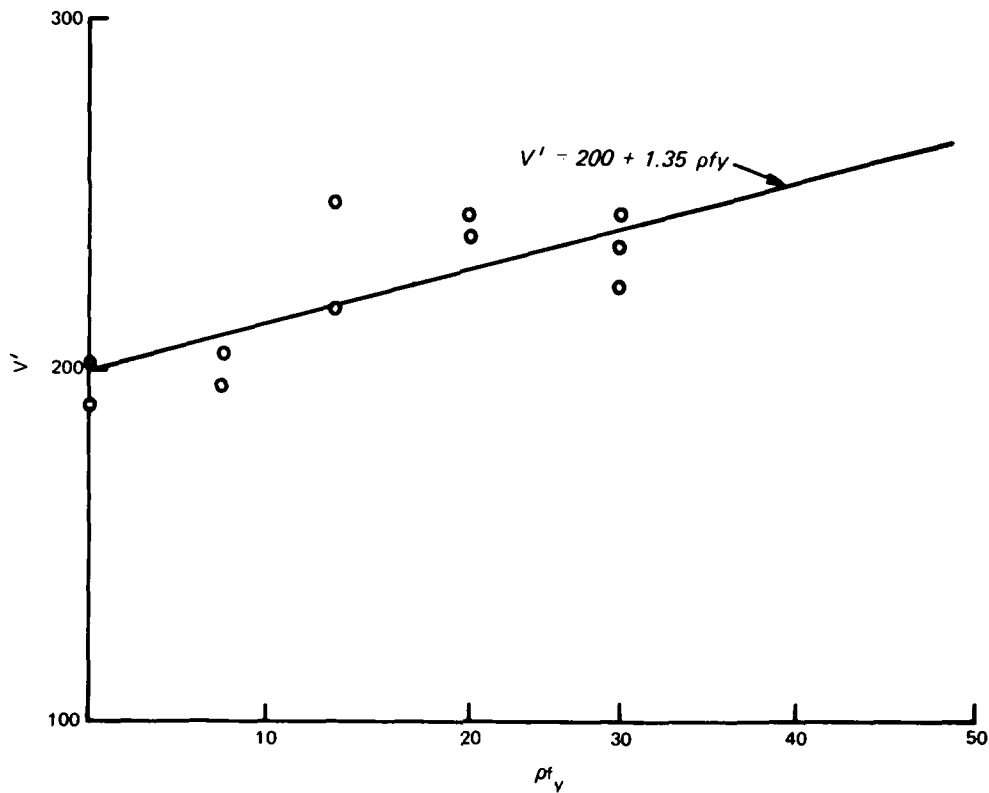
Figure 41. Relation between shear strength V' and
reinforcement parameter ρf_y

65. Based on the test data, the relationship between average ul-
timate shear stress and the value of ρf_y was derived using the least-
square fit technique:

$$V' = 200 + 1.35 \, \rho f_y \tag{5}$$

The value of the coefficient of friction obtained from the test data,
1.35, is consistent with values reported in the literature.

66. The specimens instrumented during loading experienced essen-
tially no differential movement between the old and new concretes prior
to failure with failure defined as the maximum load the block would
carry. Two distinct modes of behavior for the replacement concrete were
noted at failure. In the specimens without dowels, failure resulted

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when the top block (new concrete) dropped completely down onto the
bottom block as a result of a brittle fracture. In the specimens with
dowels (regardless of dowel size), the failure was more ductile; the
dowel was able to carry the dead load of the new concrete, thus prevent-
ing the new concrete from dropping completely onto the old concrete.
This could be of importance in a prototype structure.

67. Examination of the failure surface of several of the speci-
mens showed that failure did occur on the plane between the old and new
concretes. These surfaces show bond failures with some plucking of ag-
gregate particles from the old and new concretes. A very small percent-
age of aggregate particles was broken. One failure surface is shown in
Figure 42.
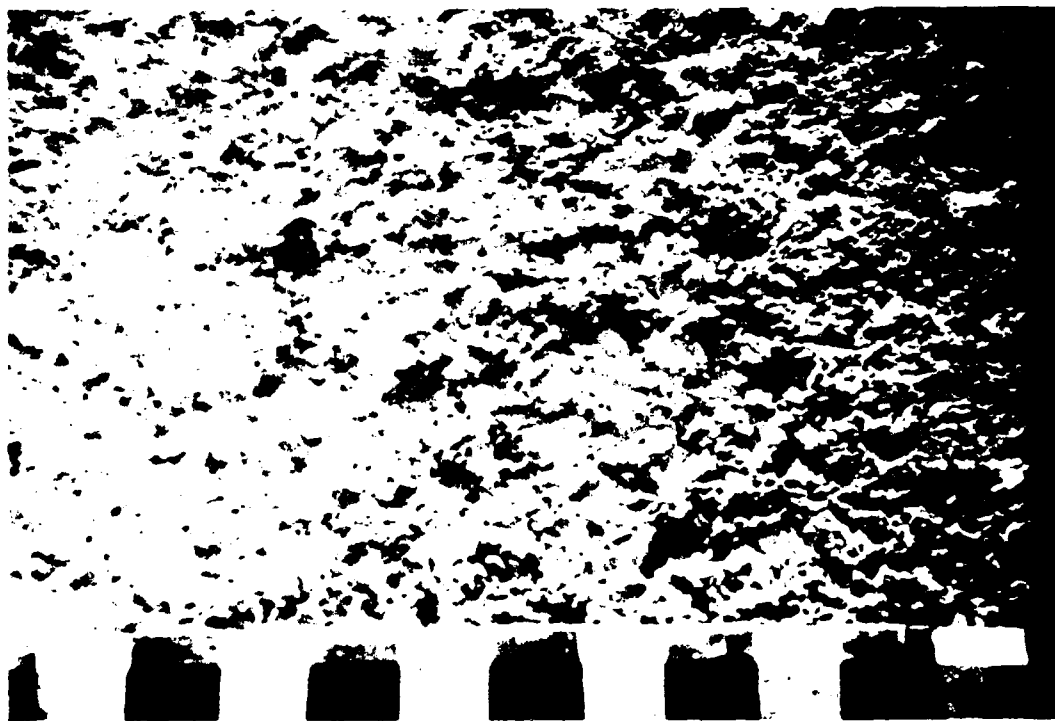
Figure 42. Close-up of failure surface of laboratory test block.
Note the bond failures and the small percentage of aggregate par-
ticles broken

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PART IV: DESIGN METHOD FOR DOWELS

Introduction

68. This part describes the development of a recommended design
method for dowels for anchoring replacement concrete to vertical lock
walls. A design example is also included. This design method is based
on the shear-friction concept discussed in Part III.

69. Laboratory test results indicate that well-bonded concrete
is relatively strong in shear transfer; however, there is always the
possibility that a crack will form at the interface because of shrinkage,
thermal stresses, or any other reason. Therefore, in the design of
dowels, a crack should be assumed to be present along the interface,
with relative displacement along the assumed crack resisted by friction
maintained by dowels across the assumed crack.

Derivation of Design Equation

70. According to the American Concrete Institute (ACI) Building
Code, ACI 318-77 (ACI 1977), the design of cross sections subject to
shear should be based on

$$V_u \leq \phi V_n \tag{6}$$

where

V_u = factored shear force at section considered

ϕ = strength reduction factor = 0.85 for shear

V_n = nominal shear strength

Based on the shear-friction concept,

$$V_n = A_d f_y \mu \tag{7}$$

where

A_d = cross-sectional area of dowel, sq in.

53

f_y = specified yield strength of dowel, psi

μ = coefficient of friction

Substituting Equation 7 into Equation 6 and solving for A_d

$$A_d = \frac{V_u}{\phi f_y \mu} \qquad (8)$$

Yield strength of dowels

71. The design shall be based on a 40,000-psi yield strength of dowel for Grade 40 and 60 steels (ASTM 1978). Steel with a yield strength in excess of Grade 60 should not be used (Liu 1980).

Anchorage length of dowels

72. The shear-friction concept is valid for conditions in which failure is attained by yielding of the reinforcement crossing the crack. Thus, dowels must be anchored in both sides of the conc te by embedment or hooks to develop yield in the steel. For the repla ent concrete, development length for the dowels in tension sh ld be c mputed in accordance with ACI 318-77 (ACI 1977). For the existing concrete, the necessary embedment length to develop yield in the steel may be determined through pullout testing using the bonding age t that will be used for the project or by using the recommendations based upon nominal dowel diameter given below (paragraph 73). If the compressive strength of the existing concrete is less than 3000 psi, the recommendations based upon nominal dowel size may not be adequate and pullout testing should be accomplished.

73. For the polyester-resin cartridges tested in the laboratory, an embedment length of not less than 15 times the nominal diameter of the dowel (paragraph 24) was found to be satisfactory. For cement grouts, Stowe (1974) also recommended an embedment length of 15 times the nominal dowel diameter. For the epoxies he tested, Stowe recommended a somewhat shorter embedment length of 10 times the nominal bar diameter. Overall, for portland cement, epoxy-resin, or polyester-resin grouts, an embedment length of at least 15 times the nominal diameter of the dowel should be satisfactory for concretes with a compressive strength of 3000 psi or greater.

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74. Pullout testing, if required, should be accomplished using a procedure similar to that given in this report. The number of dowels to be tested should be based upon the number of dowels to be installed, variations in the compressive strength of the existing concrete, and the number of different elements or monoliths in which dowels will be installed. A minimum of 3 tests per 1000 dowels to be installed is recommended, with the tests dispersed over the entire surface area which will receive dowels. Based on the laboratory and field testing, it appears unnecessary to measure bar displacement during pullout testing. Required embedment length may be determined by measuring applied pullout loads. When the dowel and bonding agent are able to resist the calculated yield load of the dowel, the embedment may be considered adequate. Due to the small number of tests recommended, all dowels tested should meet this criterion before an embedment length is selected for the project.

75. The lock wall surface will generally be very rough after deteriorated concrete has been removed; local variations of several inches from the design grade may be expected. Therefore, if dowels are installed such that all hooks are at the same grade (to facilitate later placement of vertical and horizontal steel), variations of several inches in embedment length may occur. Due consideration should be given to this problem of an uneven concrete surface when embedment length is being selected.

Coefficient of friction

76. Tests of laboratory specimens indicate the average coefficient of friction between old and new concretes is 1.35 (paragraph 65). To account for the variations expected in the field construction, a more conservative value of 1.00 should be used. This reduced value of 1.00 is also in agreement with ACI 318-77 recommendations for concrete placed against hardened concrete. To ensure that the coefficient of friction of 1.00 is attainable, all unsound, damaged, fouled, porous, or otherwise undesirable old concrete should be removed, and the old concrete surface should be clean, free of laitance, and intentionally roughened to a full amplitude of at least 1/4 in.

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Factored shear forces

77. The factored shear force V_u at the interface should be based on the following (Liu 1980):

$$V_u = 1.5V_D + 1.9V_L \tag{9}$$

where

V_D = shear force due to dead load

V_L = shear force due to all live loads

78. The vertical shear force at the interface in a typical lock wall is due primarily to the mass of the new concrete, and the vertical shear force due to live loads is usually small.

Dowel spacing

79. If the dowels are equally spaced at distance S both vertically and horizontally, Equation 9 can be rewritten as

$$V_u = \left(1.5Wt + 1.9v_L\right)S^2 \tag{10}$$

where

V_u = the factored shear force to be resisted by one dowel, lb

W = unit weight of concrete, lb/cu ft

t = thickness of the replacement concrete, ft

v_L = unit vertical shear stress due to all live loads, lb/ft^2

S = dowel spacing, ft

80. Substituting Equation 10 into Equation 8 and rearranging,

$$S = \sqrt{\frac{A_d \phi f_y \mu}{1.5Wt + 1.9v_L}} \tag{11}$$

81. If the vertical shear force due to live loads is negligible, Equation 11 can be reduced to

$$S = \sqrt{\frac{A_d \phi f_y \mu}{1.5Wt}} \tag{12}$$

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82. Substituting $\phi = 0.85$, $f_y = 40,000$ psi , $\mu = 1.00$, and W = 150 lb/cu ft into Equation 11 can provide a general equation for designing dowel spacing

$$S = \sqrt{\frac{34,000 \ A_d}{225t + 1.9v_L}} \qquad (13)$$

Maximum spacing

83. In most cases, the dowel spacing equation (Equation 13) will lead to spacings larger than those currently being used in practice, particularly if live loads are small, which is pointed out in the spacings shown in Table 13. It is believed that a practical upper limit on dowel spacing should be imposed based upon considerations of the possible failure mode and of constructibility. The objective of the failure consideration is to eliminate the possibility of brittle failure, as was seen in the laboratory shear test specimens that contained no dowels. The smallest percentage of steel evaluated in the laboratory shear tests was 0.019 percent (equivalent to No. 6 dowels at a 48-in. spacing). These specimens exhibited ductile failure. Specimens containing a smaller percentage of steel than 0.019 percent were not evaluated. From a constructibility standpoint, an upper limit seems appropriate to ensure proper positioning and support for the horizontal and vertical

Table 13

Theoretical Dowel Spacings*

Replacement Concrete Thickness ft	Dowel Spacing, ft							
	No. 3	No. 4	No. 5	No. 6	No. 7	No. 8	No. 9	No. 10
0.5	5.77	7.78	9.69	11.54	13.47	15.46	17.39	19.60
1.0	4.08	5.50	6.85	8.16	9.53	10.93	12.30	13.86
1.5	3.33	4.49	5.59	6.66	7.78	8.93	10.04	11.32
2.0	2.88	3.89	4.84	5.77	6.74	7.73	8.70	9.80
2.5	2.58	3.48	4.33	5.16	6.03	6.91	7.78	8.77

* Spacings developed by using Equation 13 and assuming no live load.

reinforcement in the replacement concrete. Based on these considerations (and minimum recommended bar size as discussed below), a 48-in. maximum spacing is recommended. This admittedly arbitrary recommendation is conservative; however, a significant reduction in the required number of dowels can be realized.

Minimum dowel size

84. Table 13 also shows that relatively small bars can be used to satisfy the requirements of the design equation (Equation 13), again assuming no live loads. As with dowel spacings, it is believed that there is a practical lower limit on bar size which must be imposed. The theory in this case is based on considerations of damage after bars are installed but before concrete is placed and on live loads possibly imposed on the dowels by construction workers during the setting or tying of outside steel. An additional consideration is the relatively low cost of the steel compared with the cost of labor involved in dowel placment. Based on these considerations, a minimum dowel size of a No. 6 bar is recommended.

Tensile stress across the interface

85. If tensile stresses are present across the interface where cracks are assumed, reinforcement for the tension must be provided in addition to that provided for shear-friction. However, the tensile stresses due to temperature and shrinkage need not be considered because these stresses are self-limiting in a cracked interface.

Design Example

Design information

86. In a navigation lock wall rehabilitation project, 12 in. of deteriorated surface concrete will be removed and new replacement concrete will be placed to return the wall to original dimension. The following design information is given.

        a. Thickness of the replacement concrete = 1.0 ft.

        b. Specified concrete compressive strength = 3000 psi.

        c. Dowel size = No. 6 deformed bars.

<u>d</u>. Design yield strength for dowels = 40,000 psi.

<u>e</u>. Vertical shear stresses due to all live loads are negligible.

<u>f</u>. Concrete cover = 4 in.

<u>g</u>. Dowels will be anchored into the old concrete with a commercially available polyester-resin cartridge system.

<u>h</u>. Existing concrete compressive strength = 3000 psi.

87. The dowel spacing can be determined from Equation 13.

$$S = \sqrt{\frac{34,000 \ A_d}{225t + 1.9v_L}} \qquad (13, \ bis)$$

where

$A_d = 0.44$

$t = 1.0$

$v_L = 0$

Thus

$$S = \sqrt{\frac{34,000 \times 0.44}{225 \times 1.0}}$$

$$= 8.15 \ ft > 4 \ ft$$

The maximum dowel spacing of 4 ft should be used.

Development length
in replacement concrete

88. In accordance with ACI 318-77, the development length ℓ_d for deformed bars in tension should be computed by

$$\ell_d = 0.04 \ A_b \ f_y \Big/ \sqrt{f_c'} \qquad (14)$$

but not less than

$$0.0004 \ d_b \ f_y \qquad (15)$$

where

A_b = area of individual bar, sq in.

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f'_c = specified compressive strength of concrete, psi

d_b = nominal diameter of bar, in.

89. Solving Equation 14 gives

$$\ell_d = 0.04 \times 0.44 \times 40,000/\sqrt{3000}$$
$$= 12.85 \text{ in.}$$

whereas Equation 15 gives

$$\ell_d = 0.0004 \times 0.75 \times 40,000$$
$$= 12.0 \text{ in.}$$

Thus, the required development length is 12.85 in., which is greater
than the thickness of the replacement concrete minus concrete cover, and
hooked dowels should be used. The equivalent embedment length ℓ_e of
a standard hook may be computed in accordance with ACI 318-77 (ACI 1977):

$$\ell_e = 0.04 \, A_b \, f_h \Big/ \sqrt{f'_c} \tag{16}$$

where
$$f_h = 360 \sqrt{f'_c}$$
Thus

$$\ell_e = 0.04 \times 0.44 \times 360\sqrt{3000}/\sqrt{3000}$$
$$= 6.33 \text{ in.}$$

The combined development length = (12-4) + 6.33 = 14.33 in. > 12.85 in.
Therefore, the standard hook may be used.

Embedment length in old concrete

90. Assuming that the polyester-resin cartridges in this example
are the same type as those tested in the laboratory, an embedment length
of not less than 15 times the nominal diameter of the dowel will be
satisfactory. Therefore, an embedment length of 11.25 in. is required.

PART V: RECOMMENDED DESIGN CRITERIA FOR DOWELS FOR ANCHORING
REPLACEMENT CONCRETE TO VERTICAL LOCK WALLS

91. Based on the discussions presented in Parts II, III, and IV,
the following design criteria for dowels for anchoring replacement con-
crete to vertical lock walls are recommended.

a. All unsound, damaged, fouled, porous, or otherwise unde-
sirable concrete shall be removed.

b. The bonding surface shall be clean and free of laitance
or other materials that could inhibit bond. If the sur-
face is smooth after removal of deteriorated concrete,
the surface shall be intentionally roughened to a full
amplitude of at least 1/4 in.

c. Deformed reinforcing bars of a size not less than No. 6
shall be used as dowels.

d. The dowel spacing shall not exceed either the quantity
computed by the following equation or 4 ft.

$$S = \sqrt{\frac{34,000 \ A_d}{225t + 1.9v_L}}$$

where

S = dowel spacing, ft

A_d = cross sectional area of one dowel, sq in.

t = thickness of new replacement concrete, ft

v_L = vertical shear stress due to all live loads,
    lb/sq ft

e. A dowel spacing exceeding the limits specified above
shall not be used unless approved by higher authority.

f. Direct tension across the interface between old and new
concretes shall be provided for by reinforcement in addi-
tion to the dowels.

g. All dowels shall be fully anchored into existing and
replacement concretes:

(1) Existing concrete. The dowel shall be embedded in a
well-cleaned hole cut with a drill that leaves the
inner surface of the hole in a rough condition.
Nonshrinking portland-cement grout, epoxy resin, or
polyester resin may be used as the bonding agent.
For concrete with a compressive strength of less than
3000 psi, embedment shall be determined by conducting

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field pullout tests beginning with a minimum embedment of 15 times the nominal diameter of the dowel. For concrete with a compressive strength equal to or greater than 3000 psi, dowel embedment shall be a minimum of 15 times the nominal diameter of the dowel unless a shorter embedment length can be justified through field pullout testing. Consideration should be given to variations in the surface of the existing concrete, after removal of deteriorated material, when embedment length is being established.

(2) Replacement concrete. The dowel shall be anchored by embedment or hooks. The required development length shall be computed in accordance with ACI 318-77 (ACI 1977).

h. Field pullout tests, if conducted, should use the dowel installation technique and bonding agent to be used for the actual project. A minimum of 3 dowels per 1000 to be installed should be tested, with the testing dispersed over the entire surface area to receive dowels. Test procedures should be similar to those described in this report. The embedment length shall be considered adequate when the applied pullout load is greater than the calculated yield load for the dowel for all tests.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

92.   The results of laboratory pullout tests indicated that for
embedment lengths less than 15D, where D is the nominal bar diameter,
the failure was generally caused by slippage along the concrete-resin
interface.   All reinforcing bars yielded when the embedment length was
greater than 15D.

93.   The polyester-resin cartridges evaluated were an effective
bonding agent for anchoring dowels in concrete, provided that adequate
embedment lengths have been obtained and that the cartridge manufacturer's
installation recommendations have been followed.

94.   The performance of dowels installed under field conditions
using the polyester-resin cartridges was acceptable.   No modifications to
the general installation procedure seem appropriate.

95.   Within the range of the percentage of steel used in the
laboratory shear transfer tests, the effect of the dowels did not ap-
pear to increase significantly the load-carrying capacity of the test
specimens.   The bond of the new to the old concrete appeared to have
been much more significant than the amount of steel present.

96.   A value of 1.35 for the coefficient of friction between new
and old concretes was developed from the laboratory shear test data.
This value is consistent with values reported in the literature.   A
more conservative value of 1.00 was included in the recommended design
equation.

97.   Two distinct failure modes were observed in the laboratory
shear tests.   In the specimens without dowels, the failure mode was
brittle with a sudden dropping of the top block (new concrete) onto the
bottom block.   In the specimens with dowels (regardless of the percent-
age of steel in the range tested), the failure was ductile, with the
dowel being able to carry the dead load of the concrete, thus preventing
the complete separation of the two elements.   The minimum reinforcement
ratio tested (equivalent to No. 6 dowels at 48-in. spacing) is apparently

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adequate to force the ductile failure mode to prevail.

98. The dowel design criteria presented are based on the shear-friction concept and are rational and simple to use. These criteria are a significant improvement over current design techniques.

Recommendations

99. The design criteria presented in Part V are recommended for designing dowels for anchoring replacement concrete to vertical lock walls.

100. Every effort should be taken to ensure adequate surface preparation of the old concrete in order to develop the shear-friction mechanism upon which the design criteria are based. These efforts will also help to ensure good bond between the old and new concretes, which will extend the life of the rehabilitation.

101. Extra effort should be required in the field to avoid or minimize impact damage or bending of dowels by work barges or other causes prior to the placement of concrete.

102. Although the dowel design criteria are based on the assumption of no bond, the importance of a good bond between the old and new concretes must not be overlooked. The use of epoxies or other bond-enhancing agents for lock wall renovation should be investigated.

103. The long-term bond behavior of the replacement concrete under extended weathering and under freezing and thawing conditions should be investigated.

REFERENCES

American Concrete Institute. 1977. "Building Code Requirements for Reinforced Concrete, ACI 318-77," Detroit, Mich.

American Society for Testing and Materials. 1978. "Standard Specifications for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement," Designation: A 615-78, 1978 Book of ASTM Standards, Part 4, Philadelphia, Pa.

Birkeland, P. W., and Birkeland, H. W. 1966. "Connection in Precast Concrete Construction," ACI Journal, Proceedings, Vol 63, No. 3.

Hofbeck, J. A., Ibrahim, I. A., and Mattock, A. H. 1969. "Shear Transfer in Reinforced Concrete," ACI Journal, Proceedings, Vol 66, No. 2.

Liu, T. C. 1980. "Strength Design of Reinforced Concrete Hydraulic Structures; Report 1, Preliminary Strength Design Criteria," Technical Report SL-80-4, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Mast, R. F. 1968. "Auxiliary Reinforcement in Concrete Connections," Proceedings, American Society of Civil Engineers, Vol 94, No. ST6.

Mattock, A. H. 1974a. "Effect of Aggregate Type on Single Direction Shear Transfer Strength in Monolithic Concrete," Report No. SM-74-2, Department of Civil Engineering, University of Washington, Seattle, Wash.

_____. 1974b. "Effect of Moment and Tension Across the Shear Plane on Single Direction Shear Transfer Strength in Monolithic Concrete," Report No. SM-74-3, Department of Civil Engineering, University of Washington, Seattle, Wash.

_____. 1974c. "The Shear Transfer Behavior of Cracked Monolithic Concrete Subject to Cyclically Reversing Shear," Report No. SM-74-4, Department of Civil Engineering, University of Washington, Seattle, Wash.

_____. 1974d. "Shear Transfer in Concrete Having Reinforcement at an Angle to the Shear Plane," Shear in Reinforced Concrete, Vol 1, Special Publication SP-42, American Concrete Institute, Detroit, Mich.

_____. 1974e. "Shear Transfer Under Cyclically Reversing Loading Across an Interface Between Concretes Cast at Different Times," Report No. SM-77-1, Department of Civil Engineering, University of Washington, Seattle, Wash.

_____. 1976. "Shear Transfer Under Monotonic Loadings Across an Interface Between Concrete Cast at Different Times," Report No. SM-76-3, Department of Civil Engineering, University of Washington, Seattle, Wash.

Mattock, A. H., and Hawkins, N. M. 1972. "Shear Transfer in Reinforced Concrete - Recent Research," PCI Journal, Vol 17, No. 2.

Paulay, T., Park, R., and Phillips, M. H. 1974. "Horizontal Construction Joints in Cast-in-Place Reinforced Concrete," Shear in Reinforced Concrete, Vol 2, Special Publication SP-42, American Concrete Institute, Detroit, Mich.

Saucier, K. L. 1974. "Laboratory Investigation of Slipform Construction for Use in Mass Concrete Structures," Technical Report C-74-3, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Stowe, Richard L. 1974. "Pullout Resistance of Reinforcing Bars Embedded in Hardened Concrete," Miscellaneous Paper C-74-12, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

U. S. Army Engineer Waterways Experiment Station, CE. 1949. Handbook for Concrete and Cement (with quarterly supplements), Vicksburg, Miss.

APPENDIX A

LABORATORY PULLOUT TEST DATA

Table A1

Laboratory Pullout Test Results, Specimen 1

6-in. Embedment Length (L/D = 8)

Pump Pressure psi	Axial Load lb	Average Dial Gage Reading in.
200	2,780	0.008
400	5,560	0.016
600	8,340	0.024
800	11,120	0.040
1000	13,900	0.069
1200	16,680	0.124
1400	19,460	
1600	22,240	

Table A2

Laboratory Pullout Test Results, Specimen 2

6-in. Embedment Length (L/D = 8)

Pump Pressure psi	Axial Load lb	Average Dial Gage Reading in.
200	2,780	0.005
400	5,560	0.019
600	8,340	0.037
800	11,120	0.090
1000	13,900	0.160
1200	16,680	
1400	19,460	
1600	22,240	

Table A3

Laboratory Pullout Test Results, Specimen 3

7.5-in. Embedment Length (L/D = 10)

Pump Pressure psi	Axial Load lb	Average Dial Gage Reading in.
200	2,780	0.006
400	5,560	0.008
600	8,340	0.010
800	11,120	0.015
1000	13,900	0.023
1200	16,680	0.038
1400	19,460	
1600	22,240	

Table A4

Laboratory Pullout Test Results, Specimen 4

7.5-in. Embedment Length (L/D = 10)

Pump Pressure psi	Axial Load lb	Average Dial Gage Reading in.
200	2,780	0.005
400	5,560	0.010
600	8,340	0.018
800	11,120	0.032
1000	13,900	0.048
1200	16,680	0.065
1400	19,460	0.103
1600	22,240	

A3

Table A5

Laboratory Pullout Test Results, Specimen 5

11.25-in. Embedment Length (L/D = 15)

Pump Pressure psi	Axial Load lb	Average Dial Gage Reading in.
200	2,780	0.004
400	5,560	0.009
600	8,340	0.018
800	11,120	0.034
1000	13,900	0.057
1200	16,680	0.085
1400	19,460	0.128
1600	22,240	

Table A6

Laboratory Pullout Test Results, Specimen 6

11.25-in. Embedment Length (L/D = 15)

Pump Pressure psi	Axial Load lb	Average Dial Gage Reading in.
200	2,780	0.005
400	5,560	0.009
600	8,340	0.012
800	11,120	0.017
1000	13,900	0.022
1200	16,680	0.028
1400	19,460	0.036
1600	22,240	0.076

Table A7

Laboratory Pullout Test Results, Specimen 7

15-in. Embedment Length (L/D = 20)

Pump Pressure psi	Axial Load lb	Average Dial Gage Reading in.
200	2,780	0.007
400	5,560	0.014
600	8,340	0.017
800	11,120	0.023
1000	13,900	0.032
1200	16,680	0.043
1400	19,460	0.058
1600	22,240	0.080

Table A8

Laboratory Pullout Test Results, Specimen 8

15-in. Embedment Length (L/D = 20)

Pump Pressure psi	Axial Load lb	Average Dial Gage Reading in.
200	2,780	0.005
400	5,560	0.010
600	8,340	0.015
800	11,120	0.020
1000	13,900	0.024
1200	16,680	0.028
1400	19,460	0.035
1600	22,240	0.050

APPENDIX B
LABORATORY SHEAR TEST DATA

B1

Table B1

Shear Test Detailed Data

No Dowel

Parameter	Sample 1	Sample 2*	Sample 3	Sample 4**
Old Concrete				
Age at testing, days	6∠		65	
Compressive strength, psi				
Cylinder 1	5390		5,540	
Cylinder 2	5520		5,680	
Cylinder 3	5020		5,430	
Average	5310		5,550	
Flexural strength, psi				
Beam 1	815		890	
Beam 2	710		870	
Beam 3	815		765	
Average	780		840	
New Concrete				
Age at testing, days	28		29	
Compressive strength, psi				
Cylinder 1	5210		4,580	
Cylinder 2	5200		4,530	
Cylinder 3	5090		4,560	
Average	5170		4,560	
Flexural strength, psi				
Beam 1	765		840	
Beam 2	790		860	
Beam 3	755		750	
Average	770		815	
Shear Test				
Total load, lb	†	155,000††	109,500	115,750
Shear stress, psi		269	190	201

   *   Same concrete as Sample 1.
  **   Same concrete as Sample 3.
  †   Sample did not fail.
††   Sample not included in averages presented in Table 12.

Table B2

Shear Test Detailed Data

No. 3 Bar (0.11 sq in.)

Parameter	Sample 1	Sample 2*	Sample 3	Sample 4
Old Concrete				
Age at testing, days	61		NA	NA
Compressive strength, psi				
Cylinder 1	6,010			
Cylinder 2	6,260			
Cylinder 3	--			
Average	6,140			
Flexural strength, psi				
Beam 1	830			
Beam 2	810			
Beam 3	835			
Average	825			
New Concrete				
Age at testing, days	29			
Compressive strength, psi				
Cylinder 1	5,210			
Cylinder 2	4,920			
Cylinder 3	5,230			
Average	5,120			
Flexural strength, psi				
Beam 1	835			
Beam 2	710			
Beam 3	715			
Average	755			
Shear Test				
Total load, lb	113,250	120,000		
Shear stress, psi	197	208		

* Same concrete as Sample 1.

B3

Table B3

Shear Test Detailed Data

No. 4 Bar (0.20 sq in.)

Parameter	Sample 1	Sample 2*	Sample 3	Sample 4
Old Concrete				
Age at testing, days	84		NA	NA
Compressive strength, psi				
Cylinder 1	6,290			
Cylinder 2	6,510			
Cylinder 3	6,600			
Average	6,470			
Flexural strength, psi				
Beam 1	890			
Beam 2	960			
Beam 3	925			
Average	925			
New Concrete				
Age at testing, days	27			
Compressive strength, psi				
Cylinder 1	5,450			
Cylinder 2	5,520			
Cylinder 3	5,460			
Average	5,480			
Flexural strength, psi				
Beam 1	810			
Beam 2	725			
Beam 3	740			
Average	760			
Shear Test				
Total load, lb	126,000	142,750		
Shear stress, psi	219	248		

* Same concrete as Sample 1.

Table B4

Shear Test Detailed Data

No. 5 Bar (0.31 sq in.)

Parameter	Sample 1	Sample 2*	Sample 3	Sample 4**
Old Concrete				
Age at testing, days	68		73	
Compressive strength, psi				
Cylinder 1	5,140		4,950	
Cylinder 2	5,240		5,060	
Cylinder 3	4,930		5,130	
Average	5,100		5,050	
Flexural strength, psi				
Beam 1	825		745	
Beam 2	770		850	
Beam 3	770		845	
Average	790		815	
New Concrete				
Age at testing, days	28		28	
Compressive strength, psi				
Cylinder 1	5,240		5,060	
Cylinder 2	5,320		4,670	
Cylinder 3	5,310		5,060	
Average	5,290		4,930	
Flexural strength, psi				
Beam 1	790		810	
Beam 2	810		805	
Beam 3	795		775	
Average	800		795	
Shear Test				
Total load, lb	141,000	+	109,000++	137,500
Shear stress, psi	245		189	239

* Same concrete as Sample 1.
** Same concrete as Sample 3.
+ Sample did not fail.
++ Sample not included in averages presented in Table 12.

Table B5

Shear Test Detailed Data

No. 6 Bar (0.44 sq in.)

Parameter	Sample 1	Sample 2*	Sample 3	Sample 4**
Old Concrete				
Age at testing, days	40		70	
Compressive strength, psi				
Cylinder 1	5,580		4,540	
Cylinder 2	5,240		5,920	
Cylinder 3	5,390		5,940	
Average	5,400		5,470	
Flexural strength, psi				
Beam 1	850		750	
Beam 2	875		795	
Beam 3	830		830	
Average	850		790	
New Concrete				
Age at testing, days	28		28	
Compressive strength, psi				
Cylinder 1	4,810		4,700	
Cylinder 2	4,870		4,900	
Cylinder 3	5,000		4,740	
Average	4,890		4,780	
Flexural strength, psi				
Beam 1	695		820	
Beam 2	685		790	
Beam 3	625		785	
Average	670		800	
Shear Test				
Total load, lb	135,000	128,000	159,000†	140,500
Shear stress, psi	234	222	276	244

 * Same concrete as Sample 1.
** Same concrete as Sample 3.
 † Sample not included in averages presented in Table 12.

Liu, Tony C.
  Design of dowels for anchoring replacement concrete
to vertical lock walls : Final report / by Tony C. Liu,
Terence C. Holland (Structures Laboratory, U.S. Army
Engineer Waterways Experiment Station) ; prepared for
Office, Chief of Engineers, U.S. Army. -- Vicksburg,
Miss. : U.S. Army Engineer Waterways Experiment Station ;
Springfield, Va. : available from NTIS, 1981.
  66, [11] p. : ill. ; 27 cm. -- (Technical report /
U.S. Army Engineer Waterways Experiment Station ;
SL-81-1.
  Cover title.
  "March 1981."
  "Under CWIS No. 31553."
  Bibliography: p. 65-66.

  1. Anchor stone. 2. Concrete construction. 3. Design.
4. Dowels. 5. Locks (Hydraulic engineering). I. Holland,
Terence C. II. United States. Army. Corps of Engineers.

Liu, Tony C.
  Design of dowels for anchoring replacement : ... 1981.
                                              (Card 2)

Office of the Chief of Engineers. III. United States.
Army Engineer Waterways Experiment Station. Structures
Laboratory. IV. Title V. Series: Technical report
(United States. Army Engineer Waterways Experiment
Station) ; SL-81-1.
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